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September 30, 1920

Engineering News-Record

Design of the 37,500-Horsepower Turbines at Niagara

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By Lynn Perry

Utility of Hardwoods for Paving Shown in Comparative Tests
By E. E. Butterfield

Strength Tests on Douglas Fir and Southern Pine
Two Articles by
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Old Earth Pressure Theories and New Test Results
By Dr. Charles Terzaghi

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ENGINEERING NEWS-RECORD

DEVOTED TO CIVIL ENGINEERING AND CONTRACTING

E. J. MEHREN, Editor

Volume 85 — NEW YORK, THURSDAY, SEPTEMBER 30, 1920 — Number 14

Using Large Size Aggregate

PROVIDED it can be readily secured so that the cost of transportation is not governing, large stone makes the most economical aggregate for mass concrete, particularly when strength necessities are so low that the leaner mixtures can be used. The main reason why stone above the standard 3 or 4 in. is not more generally used is the fear that the ordinary concrete machinery will not handle it. Nevertheless, in an increasing number of cases it is being proved that regular mixing and chuting equipment, with only a few changes to provide proper clearance, will stand up under the battering of the big stones and deliver a well-mixed concrete. The latest instance is the Stevenson dam work described on another page. Here there was an ample gravel supply handy to the site, a condition which often exists but is not always utilized, and the engineers estimate that about $250,000 was saved in the concrete cost due to the decision to use the gravel even up to the 6-in. rejections, which means some irregular pieces of larger dimensions. Furthermore, they testify that the maintenance charge and out-of-order time losses were little if any greater than with stock material. The practice is well worth consideration for other jobs.

"Direct Action" in Britain

THE COAL STRIKE threatened in Great Britain, it appears at this writing, may be averted. Owing to Lloyd George's influence it has been postponed for a week to give opportunity for further attempts at settlement. Only the wage matter is now at issue, the miners having withdrawn the demand that the domestic price of coal be reduced by the amount collected as a tax on export coal. The respite of a week is not only a triumph for Lloyd George but probably an indication that the miners' leaders realize that they would lack the support of the public in their efforts to force the Government into submission. The real issue, though the immediate questions were raised over the price of coal and wages, was never concealed. Smillie, the leader, declared that it was other than the nationalization of the mines. The miners were determined to attempt to bring about a fundamental change by force instead of through the ballot box. They had chosen "direct action" instead of the constitutional process. Though the leaders would have frowned on destruction of property, their proposed method was no less anarchic than that of Lenin or the Italian workmen. The fundamental nature of the issue was appreciated by the British public, and there seems to be no doubt that even the miners began to realize that "direct action" could not succeed without popular support. The railroad strike last year lacked public sympathy, even though the issue was not fundamental. There was certainty, therefore, that this one would have been equally unpopular. It was doomed to failure. The Government would have appealed to the country and doubtless would have been enthusiastically supported. All of which is another way of saying that the sanity which has so consistently marked the attitude of the British people is still strong. Industrial and labor conditions there are unsatisfactory. Much stress is expected before there is stability, but there is as little reason for expecting a Bolshevistic England as a Bolshevistic Canada or United States.

To Aid Train Movements

DOUBLE TRACKING of main lines from each important terminal to the first passing track and a more general use of automatic block signals are among the measures recommended by the American Train Dispatchers Association to relieve the serious congestion of railway traffic. Considerable attention has been given to facilitating operations within freight yards for the relief of this congestion, but a frequent cause of interruption in traffic movements is a delay to trains approaching or getting away from yards on single-track lines. On double-track lines with especially heavy traffic there may be an equal advantage in providing additional trackage for several miles from a terminal or division point in order to facilitate the movements of freight trains within the congested district. The advantages of automatic block signals for a similar purpose are obvious, since they permit of short blocks at comparatively small expense, while with the manual system the cost of installation and operation makes short blocks prohibitive. In the earlier days of automatic block signaling on American railways there were numerous objections from operating officials to the effect that the introduction of this system would make it impossible to handle heavy traffic. Experience very soon showed, however, that the heavier the traffic the greater were the advantages of the automatic block system. Near terminal points it is especially desirable to reduce the length of the block sections to a minimum, in order to increase the traffic capacity; the automatic system meets this condition with efficiency and economy.

Strength of Wood

UNDER QUITE unusual circumstances, data tending to overturn existing views on the bearing strength of wood on surfaces inclined to the grain are contained in two articles printed in this issue. Hitherto it has been assumed that the variation of bearing strength with angle between surface and grain could be expressed by a single general law, applicable to any kind of wood. In fact, both of the investigators who now present their test results assumed this to be the case. One of them worked with yellow pine, the other with Douglas fir and white pine, and each believed that the law indicated by his results would apply generally. But by comparing the two test series it may be seen that the two investi-

gators deduce quite different laws. Since in each case the results are satisfactorily consistent among themselves, it can only be concluded that different kinds of wood are subject to different laws in this particular phase of bearing strength. Just what laws hold for other woods than those involved in the present tests is bound to be uncertain until further experiments are made. On this account, safety would seem to dictate that in any practical application the lower of the two existing formulas, the Howe formula, should be used, at least until fuller guidance is obtained by special tests on the particular wood in question. From the standpoint of theoretical study, the case is even more interesting, for present knowledge of the mechanical action of wood seems to afford no reason for the observed difference between different varieties of wood. Yet such a reason should be determinable; in other words, if our present knowledge of wood does not explain this difference of behavior, it is very likely in error. However, before such a conclusion could be considered well founded it would be necessary to repeat the tests, arranging them in such a way as to eliminate shearing. In practical design, shear is provided for separately from surface crushing, while in the tests there was a possibility for the specimens to fail by shearing without developing the full strength of the bearing surfaces. Due to this peculiarity of the tests, their value for the present is only cautionary.

Research in Soil Mechanics

THREE LINES of attack on the problem presented by earth as an engineering material are available, and by a remarkable coincidence the past few months have brought to knowledge important attempts along each of these three lines. It is our privilege to publish one of them in the present issue. The problem involved is so important that it deserves to be ranked as the outstanding research problem in civil engineering. Vast sums of daily expenditure are affected by it.

More money and more labor are expended in earthwork or in construction depending directly on the properties of earth than in any other kind of engineering construction. Yet the field of earthwork, which should be the most fruitful of all in the application of scientific method, is the playground of raw empiricism. Dealing with more different kinds of earth than of any other material—for soils are of infinite variability—engineers are virtually unable to compare two earths or differentiate between them in quantitative terms.

The problem has been with us for decades. Its solution would mean that we can deal with earth as with other materials of construction, by predetermination of effect and by efficient proportioning of structure to service. For brevity we may call it the retaining-wall problem, since the retained bank is the classic type-example of earthwork analysis; but its real scope is vastly broader, covering all questions of condition and of cause and effect in the excavation and deposition of earth as well as in every case of contact of a mass of earth with water, earth or structures. The retaining-wall problem concerns foundations, embankments, tunnels, stream channels, dams, roads and docks.

Established earth-pressure theory has long been discredited. During the last thirty years or more a large bulk of literature on the defectiveness of the theory has grown up, and instance after instance has proved that the basis of fact on which the theory rests is so microscopically small that the theory cannot stand. In spite of this proof, however, the old formulas continue in use, simply for the reason that the destructive criticism included so little new work that the building up of an improved science has not been possible. Perhaps most of the investigators were blinded by the brilliancy of the mathematical devices used in developing the old theories; at any rate, they failed to see that a new start should be made, by measuring and weighing elementary facts about earth and its behavior. Making this new start is the work of the early future, by all prognostications.

What is to be the direct objective of the study of earth as an engineering material? The infinite variability of the material brings this question to the front. Shall a true and complete theory of the mechanics of earth be aimed at, or ought we seek for a classification of earth into a limited number of soil groups? Probably the latter is the more immediate objective.

For, while it is possible that we may succeed in discerning and measuring all the pertinent properties of soils, and in developing from these data of fact the mechanics of earth so completely as to equip us for handling any practical problem by advance computation, it is unlikely that such outcome would be of much practical utility. The methods would necessarily be highly complex, and they would be unlikely to give more than tentative results because the material encountered in the work will always be variable, and usually cannot be known in advance. The whole aspect of the problem as it appears at present makes it probable that soils can be dealt with by relatively simple special methods devised for each of several groups of earth types, with sufficient quantitative differentiation within the group to suit the range of properties covered. For this purpose we may study the kinds of earth by groups, and assemble and average the data for each. But to render this possible the properties of earth and how they vary must be searched out, their similarities and their effects on the engineering of earth must be determined, and from this knowledge a classification into groups worked out.

As to the three available lines of attack on the general problem: The first, which comprises formulating a new hypothesis of earth action and comparing its results with experience-data, is represented in newest form by a paper of H. G. Moulton read last February before the American Institute of Mining Engineers. Mr. Moulton's paper, a notable contribution to earth-pressure literature, is entitled to careful study. But however valuable his hypothesis may prove for retaining walls and trench sheeting, we may safely assert that with respect to the general earth problem this line of attack has been exhausted, and that its failure indeed was inevitable in view of the hopeless lack of data on earth properties. For the time being it may be written down as an axiom that constructing new hypotheses of earth pressure is wasted effort. Further, earth pressure is not the only point of inquiry; there are many other problems in earthwork.

Second is the statistical method of study, used by the foundation soils committee of the American Society of Civil Engineers. A progress report of this committee appears in last month's *Proceedings* of the society (August, 1920). The committee has attacked the problem of classification by grain-size grading, leading up to a basis for statistical collection of observed facts on soil behavior, forecast by a synopsis outlined in the report

mentioned. The crux of the matter is whether a valid classification will be achieved. As yet the outcome cannot be foreseen, but the effort made is of great promise.

Dr. Terzaghi's article in this issue represents the third method: research into the elementary facts of earth mechanics, as a preliminary to either theory or classification or both. In its first stages such work is a study to determine not laws but mere facts of earth action, from which characteristic properties may be discerned. It is a groping search for the quantities which are to be measured, a search which in itself must be conducted by measuring, weighing and comparing to detect regularity of behavior. The slow procedure of investigating the nature of an unknown thing must be followed, necessarily. This is truly starting from the beginning, with the ultimate purpose of setting up scientific knowledge based on the behavior of the material and discovery of the quantities determining that behavior.

What is published in this issue represents a small part of the summing up of several years' laboratory study. How far the total of work will lead is not yet apparent. It may be expected that in the course of the study each discovery of a lawful relation between cause and effect will bring us nearer the goal of grouping or co-ordinating soils, and defining the groups or types by characteristic properties subject to measurement. But even the present article succeeds in developing a new group of facts, and therewith it heralds the opening of an avenue of progress which promises to lead on toward a more definite knowledge of earth.

Influence of Price Declines

PRICE REDUCTIONS were so marked last week as to draw general attention to them. The newspapers display the matter prominently, and this will aid in encouraging the downward movement. As a rule, when price-cutting becomes general buying holds off in the hope of still further reductions. Under present circumstances, with prices from 100 to 300 per cent higher than before the war, we may expect still further declines.

Drops so far recorded range from 20 to 33 per cent. These are substantial on lines that increased only 100 per cent, for on them a 50-per cent reduction brings the price back to normal. A 33⅓-per cent reduction knocks off 66⅔ per cent of a 100 per cent increase.

Fortunately, the situation is being looked on complacently. The bankers, particularly, feel that the situation is well in hand, that the lowering of prices will bring a healthier business condition, that the decline was inevitable, and that, on account of its orderly procedure and the soundness of fundamental conditions, panic is not to be feared.

There are those, of course, who are pinched, or whose interest is served by a highly speculative market at high prices, but their voice, fortunately, gets little attention. They argue that the cessation of buying is reducing production and that the present lessening of demand and the shortening of supply will surely be followed by heavier demand and another rise later on. This view is based on the assumption that the buying spirit of the public last year was normal and will reassert itself in last year's intensity. Such a reassertion is quite doubtful. Last year's buying was decidedly abnormal. It was consequent upon the newness of high wages. Now the novelty has worn off, and there is thought again of laying something by and of getting one's money's worth in purchasing. Buying is more intelligent, and the attitude is not likely to change.

Of course, there is some limit to the non-buying movement, but what that limit is no one knows. It is, per capita, at some point well below the standard of 1913, for no one will contend that then there was any large degree of self-denial. In other words, there is a possibility, at least, of a very greatly curtailed demand before price can tempt the public into the market on the 1919 scale.

That is true even of housing—admittedly very short. People can get along with poor accommodations. In fact, they are getting along with them. In every stratum the tendency is toward smaller homes—more economical to rent and to operate. The cry is not so much against the character of accommodations as against the high rentals.

With marked possibilities of economy and an intelligent buying spirit among the people, we doubt whether a reaction against the present tendency is likely to come, or will be of importance if it should. We are on a more intelligent and discriminating buying level. Under such circumstances the markets are more sensitive to price increases than before. Even slight reactions will drive away those who have been attracted by the declines.

This greater discrimination in buying will naturally affect most the luxury market; this is already apparent in the slump in the automobile business. The consequences are not pleasant for luxury trades, but the business of the country, as a whole, will be on a sounder footing. The men released from luxury industries will go into the production of staples as fast as the price reductions in the latter stimulate the demand.

Fundamentally, conditions are good. The crops this years are excellent and the banking situation well in hand. Furthermore, there is such a shortage in all lines that when sane demand, based on a lower price scale, becomes operative, there must set in a long era of active business. In construction work there is, depending on the specialty, a shortage of three to six years based on pre-war business. That whole shortage need not necessarily be made up, but certainly a good part of it will be. While, therefore, engineering lines are "slow" now, there are years of solid prosperity ahead.

Building material prices, as yet, show no general recession. In fact, apparently good demonstrations have been made, based on the figures, that there will be no decline, but such demonstrations have been made on the assumption that the whole of the shortage will be made up and be made up quickly. It is our view, though, that the public will choose its time regarding building operations and that the reductions in other lines will stimulate the holding off of building activity for lower price levels. Each drop, naturally, will bring into the market a group whose necessities are such that the reductions tempt them into the market.

The problem, obviously, is not a simple one. One can get snarled up in considerations of the effects of transportation and coal shortages. They have a bearing—an important one—but the outstanding and controlling influence is the disposition of the buying public. They are returning to sanity in their purchasing. If they continue in their present temper, the declines will continue, and if those declines are orderly, without panic, the situation, in view of the shortage which promises years of steady work when prices are attractive, should be considered encouraging.

Old Earth-Pressure Theories and New Test Results

First Report on a Research Into Pressure in Granular Masses — Errors of Old Theories Due to Faulty Assumptions—Test Apparatus—Lateral Pressure Ratio—Slip Phenomena—Relation to Density

By Dr. Charles Terzaghi
Robert College, Constantinople, Turkey

The earth-pressure theories of Coulomb, Rankine and their successors still occupy today a very important place in engineering literature and in engineering practice. But the arguments accumulated during the last forty years against their validity are so powerful that these theories are ripe for being definitely discarded and replaced by more up-to-date tools.

If a retaining wall yields, a slip occurs in the backfilling, and between the wall and the backfilling a wedge-shaped body of material moves downward. The wedge slides, it is said, and the slip is considered as the only characteristic effect of the breakdown of equilibrium of the mass of sand. If the retaining wall is entirely removed, the surface of the front part of the backfilling forms an even talus of a certain angle, called the angle of repose. As the slightest increase of the slope of this talus causes a movement of sand grains, it is assumed that the tangent of the angle of repose must be identical with the coefficient of internal friction of the mass. These ideas form the foundation of the earth-pressure theories mentioned, and as long as they are considered to be correct the validity of the earth-pressure theories cannot be seriously doubted. But a number of doubts arise when the facts of earth behavior are considered. Here are three, for example:

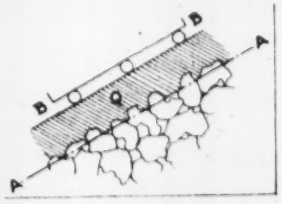


FIG. 1. ILLUSTRATING SURFACE FRICTION OF SAND AND EXTRA RESISTANCE DUE TO CHANGE OF BULK

1—If a little trapdoor located at the bottom of a layer of thoroughly compacted sand yields gradually, the stresses in the sand exceed very considerably the value determined by the tangent of the angle of repose, before equilibrium breaks down. And breakdown does not start along the planes where the tendency to slide is supposed to be a maximum, but simply in those spots where the grains have the best opportunity to move, i.e., immediately above the yielding door. Fig. 1 may explain this fact. *AA* is the even surface of a compacted sand mass, loaded with a slab *Q*, whose surface of contact is supposed to be very rough. If *Q* simply rests on top of *AA* and the slope of *AA* is gradually increased, a slip will occur as soon as the inclination of *AA* exceeds the angle of repose That is the ordinary slip of the earth-pressure theory. But if a guide *BB* parallel to *AA* be applied, such that the body *Q* cannot move except strictly parallel to *AA*, the inclination of *AA* can exceed the angle of repose without any slip occurring. This is simply because the grains cannot turn. Fig. 2 shows at (*b*) the structure of a well-compacted mass of sand. The grains are interlocked in a very complicated way and the grains cannot change their mutual position without passing through a state in which their association is less intimate, as indicated at (*a*). The less intimate the association, the greater is the volume of the voids. Increase of the volume of voids means an increase in bulk. Therefore no slip can occur except in those cases where the mass has an opportunity for lateral expansion, as in the case of backfilling behind a yielding retaining wall or in those cases where the structure of the sand has already been loosened by *preceding* breakdown of the equilibrium, as in the case of the yielding trapdoor. This leads us to suspect that the slip is not an essential but an incidental event.

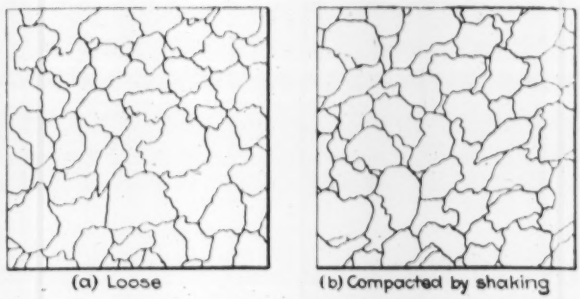


FIG. 2. LOOSE AND COMPACT STRUCTURE OF SAND (Diagrams traced from aluminum disc apparatus)

2—The formation of a talus at the angle of repose is preceded by violent movements of the sand grains; the peripheral parts of a heap of sand consist therefore of an extremely loose and unstable aggregate of grains. How is it possible to consider the statical properties of the peripheral parts of the mass as being identical with those of the compacted backfilling? Darwin observed[1] that with quartz dust loosely poured into the space behind the wall (angle of repose 35 deg., volume of voids 44.7 per cent) the first slip occurred at a wall pressure only two-thirds that given by the Rankine formula for 35-deg. angle of repose, while in the same material thoroughly compacted (volume of voids 40.9 per cent) the experiment showed a pressure about half that of the formula.

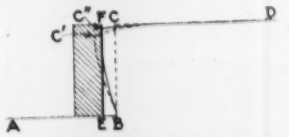


FIG. 3. HOW A SOLID BODY WOULD DISTORT IN PRESSING AGAINST A YIELDING RETAINING WALL

3—Earth-pressure theories consider earth as a solid, homogeneous material. How is it possible to discuss the lateral pressure of such a mass against an obstacle without considering its elastic deformations? In Fig. 3, representing a cross-section through a solid body uniformly loaded on *CD*, if the body could expand freely its cross-section would assume the shape *BC'D*. But its lateral expansion is partly prevented by the vertical back *EF* of a retaining wall. It is obvious, then, that the pressure acting against the wall depends in part on the distance *EB* over which the wall has yielded. An earth-pressure theory which ignores the elastic

properties of the backfilling must necessarily ignore such important relations.

The fundamental assumptions of the traditional earth-pressure theories cannot, in fact, stand even superficial examination. The fundamental error was introduced by Coulomb, who purposely ignored the fact that sand consists of individual grains, and who dealt with the sand as if it were a homogenous mass with certain mechanical properties. Coulomb's idea proved very useful as a working hypothesis for the solution of one special problem of the earth-pressure theory, but it developed into an obstacle against further progress as soon as its hypothetical character came to be forgotten by Coulomb's successors.

The way out of the difficulty lies in dropping the old fundamental principles and starting again from the elementary fact that the sand consists of individual grains. This method of dealing with the problem was first tried by Couplet, in 1727, forty-seven years before Coulomb published his theory, but Couplet failed. In 1916 and 1917 Professor Skibinsky of Lemberg[1] published a series of articles on the pressure exerted by a mass of spheres. Investigations of this kind are apt to be based on certain assumptions concerning the arrangement of the spheres, and the investigations result therefore in rather complicated formulas of limited value. In 1917 the author, who had no knowledge of Professor Skibinsky's publications, started to investigate along the same lines. But in the attempt to check theoretical results by observations he became convinced that earth pressure is an experimental rather than an analytical problem, and that we ought first of all to learn the physical facts of earth behavior.

Statical and Frictional Resistance

At depth t below the surface of a liquid, its pressure p against a vertical wall is, per unit of area, $p = tw$, where w is the weight of unit volume. The same relation exists between the depth below the surface and the lateral pressure of a mass of perfectly smooth spheres of equal size, in case the volume of voids of this mass is a maximum. The side pressure p' of any other regular assemblage of smooth spheres may be greater or less, and can be expressed by the equation

$$p' = ktw \qquad (1)$$

and k may be as small as 0.245. The cause of the difference between p and p' will be called the *statical resistance*, because it exists independently of any frictional resistances acting at the surfaces of the spheres. The statical resistance is merely the effect of the weight of the individual grains transmitted by their neighbors to the wall of the confining vessel.

If the surfaces of the smooth spheres suddenly become rough, the lateral pressure p' remains unchanged. And if the wall yields, the effect of the roughness of the surfaces develops but gradually, because the full development of the frictional resistances requires a certain amount of preceding deformation of the spheres. The resistance which causes the difference between the lateral pressure of a mass of smooth spheres and a similar mass of rough spheres at a given amount of yield of the wall will be called the *frictional resistance*.

Statical and frictional resistances co-operate in a rather complicated way, and their relative influence on the intensity of the earth pressure depends on how much the wall has yielded. During the first phase of the yielding process, i.e., from the moment when the wall starts to yield until the moment when the frictional resistances are fully developed, the statical resistance remains practically unchanged, while the effect of the frictional resistances increases. Thence, until final breakdown of equilibrium, the mutual position of the sand grains changes. The corresponding movements of the grains have been called by the author *intergranular movements*. The statical resistance increases, and simultaneously the frictional resistance increases, in spite of the fact that the total friction between the grains remains constant. Final breakdown of equilibrium takes place as soon as statical and frictional resistances reach a maximum. During the first phase an additional yield of the wall is immediately followed by a corresponding change in the intensity of the earth pressure, while during the second phase the change in the intensity of the earth pressure is in addition a function of the time. In practice the first phase passes gradually into the second one.

With these three elements—statical resistance, frictional resistance and amount of yield—a fourth must be taken into consideration, the elastic properties of the grains. Since it seemed doubtful whether the laws which determine the pressure of a mass of spheres against a yielding wall strictly apply to a mass of grains of irregular shape, it was decided to investigate by experiment the details of the relations discussed.

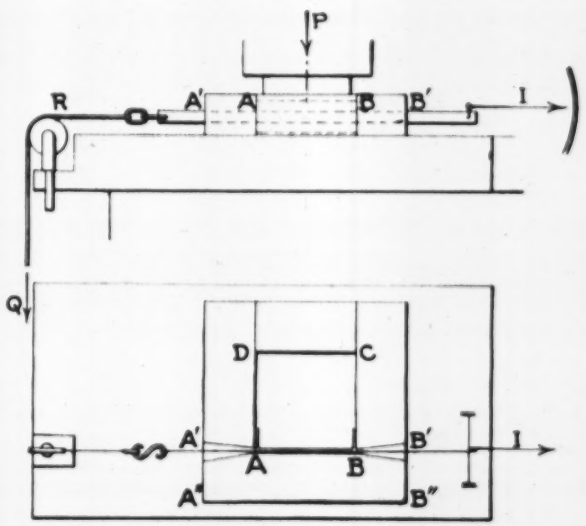


FIG. 4. APPARATUS FOR DETERMINING "EARTH PRESSURE AT REST"

Methods of Experiment Adopted

Since earth pressure against a retaining wall depends on the yielding of the wall, the leading idea in planning the experimental methods was to exclude strictly every deformation of the sand mass except the one whose effect was to be investigated.

Earth Pressure at Rest—For measuring the intensity of earth pressure against a perfectly unyielding wall, the *earth pressure at rest*, the device Fig. 4 was used. In this, one inside face *AB* of a stiff frame *ABCD*, Fig. 4, is provided with three horizontal steel tapes, whose flat surfaces rest against the face of the frame; upper and lower are fixed, while middle tape passes.

through the funnel-shaped openings AA' and BB' in the sides of the frame, being separated from the other two tapes, which serve as guides, by about 0.1 mm. Within the space AB both sides of the set of tapes are covered with paper, so that the movable tape has no contact with the sand occupying the space $ABCD$. The paper was carefully selected with a view to finding a material whose friction in contact with steel is fairly proportional to the surface pressure. It was extended over a part of the sides AD and BC, to prevent sand or dust penetrating the space between paper and tape. The frame being filled with sand, the surface was covered with a square slab S (length of side 5 mm. smaller than the frame) and subjected to pressure in a testing machine. From the front end of the movable tape a string passed over a pulley R to a loading device, while the rear end was set against the short arm of the indicator of a deflectometer I. The intensity of the pressure exerted by the sand against AB was calculated from the force Q required to overcome the friction of the tape.

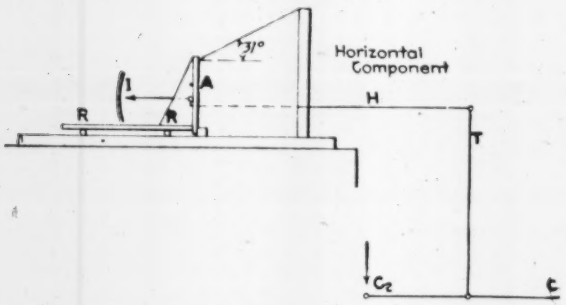


FIG. 5. RETAINING-WALL APPARATUS TO MEASURE EARTH PRESSURE UNDER LARGE DISPLACEMENTS

Even this primitive apparatus furnished very satisfactory results and it seems advisable to attempt further improvements.

Horizontal Component—Measurement of the horizontal component of the earth pressure against a retaining wall which yields in a horizontal direction was carried out thus: The retaining wall A, Fig. 5, rested on an accurately made pair of rollers R. Direct connection was made between the retaining wall and the tongue T of a weighing scale, and, while gradually reducing the resistance of the wall by loading the cup C_2, the observer noted the movements of the wall as shown by the indicator I. Similar apparatus was used for measuring the intensity of the earth pressure against battered walls.

While almost all of the older experiments were made with very fine sand, the author worked mostly with coarse sands (size of grains 2-3 mm.). It is easier to produce a fairly homogeneous backfilling with a coarse sand than with a fine sand, and the retaining wall can yield until the equilibrium of the sand breaks down without any sand entering into the space between the yielding wall and the edges of the box. It can be proved that, with equal volume of voids and similar arrangements of grains, coarse and fine materials exert the same pressure, unless the size of the grains is smaller than about 0.01 mm., in which case certain surface effects become sensible.

Vertical Component—Measurement of the vertical component of the earth pressure against a retaining wall which yields in a horizontal direction was accomplished in the following way: The retaining wall in

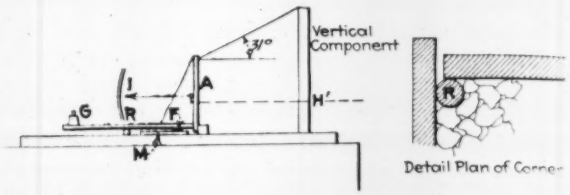


FIG. 6. MEASURING THE VERTICAL COMPONENT

Fig. 6 was supported on a single cylindrical roller R. The bottom of the front end of the wall was provided with a steel button F resting on a ground-glass surface M; weights at G balanced the wall over the roller, so that the button just touched the glass surface. Calling H the horizontal component, V the vertical component of the earth pressure and f the coefficient of friction between steel and glass, the experiment furnishes the value H'.

$$H' = H - Vf, \text{ or } V = \frac{H - H'}{f} \qquad (2)$$

The value H is known from the measurement described just above. The value of f for the friction of steel on glass is remarkably constant.

The method is an indirect one, but it excludes an important source of error common to almost all of the older methods. No pull is exerted in a vertical direction by the measuring device on the retaining wall and the measurement of the vertical component takes place while the wall yields merely in a horizontal direction, i.e., the measurements of the horizontal and of the vertical component are made under exactly the same kinematic conditions, which is the essential point.

Passive Earth Pressure—In investigating passive earth pressure the method of measuring the active earth pressure was followed except that the retaining wall was forced forward by the scale loading. At the start of every experiment the joints between the wall and the sand-box were open, and in order to prevent the sand grains from clogging the open space, the corners between the wall A and the edges of the box were provided with vertical glass rollers R, as shown in Fig. 6.

Observing Intergranular Movements—An ideal cross-section through a granular mass was obtained by cutting a great number of pieces of sheet aluminum to the shape of the enlarged cross-sections of natural sand grains, and inclosing these pieces within a frame of variable width between two parallel glass sheets spaced apart a trifle more than the thickness of the pieces. By properly manipulating the frame and gradually changing its width, the natural arrangement of the grains, the details of the movements in the sand mass and the increase of the volume of voids during such movements could be studied. The tracings for the two sketches in Fig. 2 were obtained by means of such apparatus.

The lateral pressure at rest, i.e., the pressure at the beginning of the first phase, was determined by means of the apparatus Fig. 4. In order to check the result the following method was tried: A layer of sand 5 cm. high was set under pressure. Through the center of this layer passed a horizontal steel tape with the flat side standing vertically, and at different vertical pressures the frictional resistance between the tape and the sand was measured. Then the same experiment was repeated with the flat side of the tape in a horizontal

position. The ratio between the force required to overcome the frictional resistance between the tape in the two positions at the same vertical pressure is equal to the ratio between the horizontal and the vertical pressure in the mass. In every series of experiments, the pressure was gradually raised from 0.8 kg. to 4 kg./cm.², then the load was entirely removed and gradually applied again. Both methods furnished almost identical results. It may be convenient to recall that 1 kg./cm.² is nearly equal to 1 ton/ft.²

The ratio between the horizontal and the vertical pressure was found to be independent of the absolute value of the pressures, and equal to 0.42. The angle of repose of the sand was 35 deg. With this angle, Rankine's method furnishes 0.271 and Rebhann's method 0.215, values which are 35.5 per cent and 48.8 per cent smaller than the observed value. Particular stress is laid on the fact that the preceding compression of the sand up to 4 kg./cm.² had no effect whatever on the results of the tests.

Between the beginning of the first and the beginning of the second phase, the earth pressure decreases with the distance that the retaining wall yields.

Earth Pressure During the Second Phase

Let w be the specific gravity of the sand, h the height of the retaining wall, so that $\frac{1}{2}wh^2$ would be the horizontal pressure exerted by a liquid of the same specific gravity against unit length of the wall; $H = \frac{1}{2}kwh^2$ is the horizontal component of the corresponding pressure of the sand, and $s = eh$ the distance that the retaining wall yields. Then Fig. 7 represents the relation between k and e for three typical cases. The results were obtained

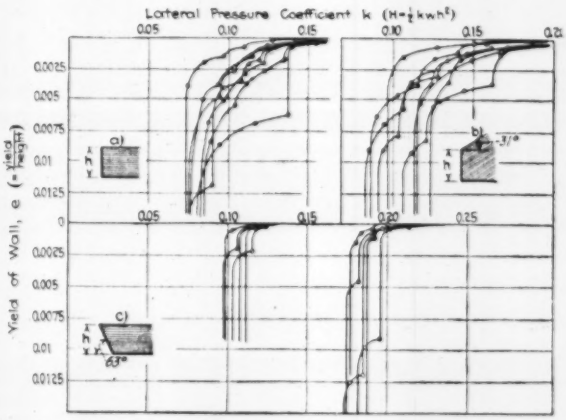


FIG. 7. TEST CURVES OF EARTH PRESSURE DURING SECOND PHASE

by means of the apparatus Fig. 5. In each figure there is introduced a diagram cross-section through wall and backfilling.

Curves (a) represent coarse river sand, size of grains 2 to 3 mm., specific gravity of grains 2.56, volume of voids 42.2 per cent, poured into place in horizontal layers. Curves (b) represent the same sand poured in layers parallel to the inclined sand surface. Curves (c) apply to a wall sloping forward at 63 deg., the sand poured in horizontal layers; the left-hand set of lines gives the values $H - 0.39V$ which were furnished by the apparatus shown in Fig. 6. Attention is called to the characteristic difference in shape between curves (a) or (b) and curve (c), the latter instancing the breakdown of equilibrium of the backfilling almost without any preceding slip. The mechanics of this process will be discussed another time.

The diagrams show that in an advanced state of the second phase the retaining wall yields only by jerks. These jerks are caused by a temporary increase in the volume of the backfilling, which takes place at every slip.

Among the different values assumed by the earth pressure during the second phase, the following three can be considered as characteristic: (1) The pressure H_1 at the beginning of the second phase; this acts at the moment when the yielding of the retaining wall becomes sensible and proceeds in a greater proportion than the earth pressure decreases. As the limit between

TABLE I. MEASURED PRESSURES OF SAND AGAINST VERTICAL RETAINING WALL

No.	Material of Backfill	Surface of Backfilling	Method of Filling the Sandbox	Volume of Voids, per Cent	Coefficient of Liquid Pressure — First Phase k_0	Second Phase k_1	Second Phase k_2	Second Phase k_3
1	Sand (*a*)	Horizontal	Poured in horizontal layers	42 2	0 42	0 162	0 130	0 079
2	Sand (*a*)	Horizontal	Poured in layers dipping toward wall	42 2	0 42	0 168	0 134	0 083
3	Sand (*a*)	Horizontal	Poured in layers dipping toward the backfill	42 2	0 42	0 140	.....	0 075
4	Sand (*a*)	Horizontal	Poured in horizontal layers, layers rammed	40 0	0 42	.....	.....	0 060
5	Sand (*a*)	Horizontal	Compared by stirring and shaking	39 8	0 42	.....	.....	Very small
6	Sand (*a*)	Rising at 31°	Poured in layers parallel to the surface	42 2	0 77	0 212	.....	0 106
7	Sand (*a*)	Dipping at 31°	Poured in layers parallel to the surface	42 2	0 25	0 123	.....	0 068
8	Sand (*b*)	Horizontal	Poured in horizontal layers	47 0	0 42	0 200	0 183	0 125
9	Sand (*b*)	Rising at 31°	Poured in layers parallel to the surface	47 0	0 77	0 270	.....	0 177
10	Sand (*c*)	Horizontal	Poured in horizontal layers	.....	0 42	.....	0 140	0 126
11	Sand (*d*)	Horizontal	Poured in horizontal layers	44 7	0 42	.....	0 180	.....
12	Sand (*d*)	Horizontal	Poured in layers dipping toward the wall	44 7	0 42	.....	0 189	.....
13	Sand (*d*)	Horizontal	Poured in layers dipping toward the backfill	44 7	0 42	.....	0 165	.....
14	Sand (*d*)	Horizontal	Compacted by stirring and shaking	40 9	0 42	.....	0 132	.....
15	Sand (*d*)	Rising at 35°	Poured in layers parallel to the surface	44 7	0 81	.....	0 291	.....
16	Beans	Horizontal	Poured in horizontal layers	.....	.....	.....	.....	0 121
17	Beans	Horizontal	Placed in horizontal layers, layers rammed	.....	.....	.....	.....	0 093
18	Beans	Horizontal	Compacted by shaking	.....	.....	.....	.....	Very small

Sand (*a*): Coarse beach sand, grains rough, mostly quartz and feldspar. Specific gravity of grains 2.56, size of grains 2-3 mm. Angle of repose 35°.
Sand (*b*): Fine beach sand, grains like those of (*a*), size of grains ½-1 mm. Angle of repose 34°.
Sand (*c*): Fine river sand, grains mostly quartz, somewhat transparent, surfaces perfectly smooth, edges partly sharp, partly rounded. Specific gravity of grains 2.52, size of grains ½-1 mm. Angle of repose 34°.
Sand (*d*): Flint powder (washed and sieved road scrapings from a flinty country. Angle of repose 35°.
Beans: 7mm. long, 5mm. thick, angle of repose 33°.

Intensity of pressure "at rest," $I_0 = \frac{1}{2}k_0wl_2$
At beginning of second phase, $I_1 = \frac{1}{2}k_1wl_2$
At first slip, $I_2 = \frac{1}{2}k_2wl_2$
At breakdown of equilibrium, $I_3 = \frac{1}{2}k_3wl_2$

Remarks—All experiments were made by the author except Nos. 11-15, which are reprinted from G. H. Darwin, "On the horizontal thrust of a mass of sand," Exc. Min. Proc. Inst. Civ. Eng. Vol. LXXI, 1883. As later experiments of the author showed that the intensity of the earth pressure against a yielding wall increases somewhat with time, it may be stated that the time between the beginning of every experiment and the breakdown of the equilibrium of the sand was about 30 minutes. Every series of experiments was repeated at least eight times. Where the experiments led to considerably diverging results no average was taken and the space for the coefficient is left blank in the table. At the time the experiments were made, the method for determining k_0 was not yet known. The value k_0 was taken from the text and introduced into the table for the purpose of comparison. The value k_0 in Nos. 6, 7, 9 and 15 were approximately computed on the basis of $k_0 = 0.42$ for backfilling with a horizontal surface. The considerable differences between the intensity of the earth pressure of coarse sand, fine sand and dust are due to the fact that the coarser the sand, the denser is the backfilling, under identical methods of filling the sand box.

The diagrams show that in an advanced state of the second phase the retaining wall yields only by jerks. These jerks are caused by a temporary increase in the volume of the backfilling, which takes place at every slip.

Among the different values assumed by the earth pressure during the second phase, the following three can be considered as characteristic: (1) The pressure H_1 at the beginning of the second phase; this acts at the moment when the yielding of the retaining wall becomes sensible and proceeds in a greater proportion than the earth pressure decreases. As the limit between

the first and the second phase is not sharply defined, the intensity of H_1 can be only roughly estimated. (2) The pressure H_2 acting during the first slip (the first slip appears in the diagram as the first sharp break in every curve). (3) The pressure H_3 at complete breakdown of equilibrium.

Table I contains a series of average numerical values of coefficients for H_1, H_2 and H_3.

Direction of the Pressure

Some indications of the direction of the earth pressure at rest were obtained by using the frame shown in Fig. 4, with an adjustment for preventing the dust from penetrating the space between the bottom of the frame and the base on which the frame rests. The frame was filled with sand and put under pressure. The steel tape was passed underneath the part $A'B'A''B''$ of the bottom of the frame between two sheets of paper and received a pressure equal to half of the friction between the sand and the inside of the frame. The angle between the direction of the earth pressure and the normal to the inside of the frame was computed from the frictional resistance acting between the tape and the papers. Its values for different vertical pressures are plotted in diagram Fig. 9. The angle increases with the vertical pressure, which in the opinion of the author is a consequence of the increasing effect of the sand surface being covered with a slab. For zero pressure the diagram indicates an angle of about 5 deg.; this seems to be the angle applying to a sand layer with free surface.

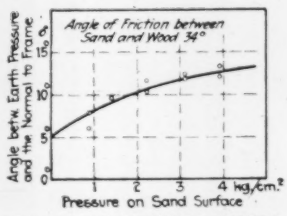

Angle of Friction between Sand and Wood 34°
Angle betw. Earth Pressure and the Normal to Frame
Pressure on Sand Surface
kg/cm.²

FIG. 8. MEASURED DIRECTIONS OF THE RESULTANT PRESSURE

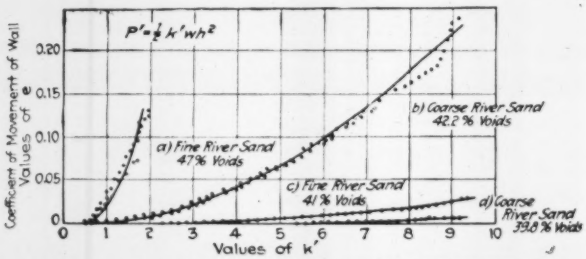

P'=½k'wh²
Coefficient of Movement of Wall
Values of e
a) Fine River Sand 47% Voids
b) Coarse River Sand 42.2 % Voids
c) Fine River Sand 41 % Voids
d) Coarse River Sand 39.8 % Voids
Values of k'

FIG. 9. TEST CURVES FOR PASSIVE PRESSURE OF FOUR MATERIALS

The values assumed by the angle during the second phase were obtained by experiments with the apparatus Fig. 6. The backfilling had a free surface. Some of the values are compiled in Table II.

TABLE II. OBSERVED ANGLE OF RESULTANT PRESSURE WITH NORMAL TO WALL

No.	Description	Sand (a)	Sand (b)	Beans
1.	Wall with vertical back, surface of backfilling horizontal, poured in horizontal layers	5°0′	2°22′	3°16′
2.	Like 1, but surface rising at 31°, poured in layers parallel to the surface	6°32′	1°35′	
3.	Like 2, but surface dipping at 31°, poured in layers parallel	3°58′		
4.	Like 1, but back of the wall dipping toward backfilling at 63°	19°33′	19°44′	

The results of the experimental investigation lead to the following conclusions:

During the second phase the pressure acts practically perpendicular to the back of vertical or battered walls, whether the surface of the sand is horizontal or not. In the case (*c*) of Fig. 7 (wall sloping forward), the direction of the earth pressure forms with the normal to the wall a certain angle, depending on the slope of the wall; with increasing slope the angle increases, reaches a maximum somewhat smaller than the angle of friction between the sand and the wall, and then decreases again.

"Earth pressure at rest" is in every case directed at an angle with the surface against which it acts, but this angle, too, is only a fraction of the angle of friction between the sand and the wall.

Passive Earth Pressure

When a force acting against the wall tends to compress the backfilling, the resistance of the backfilling increases in direct proportion to the distance of compression until the frictional resistance which acts between the surface of contact of the sand grains assumes its maximum value. That is the first phase of passive pressure. It seems to be of less importance than the first phase of the active pressure.

The second phase starts with intergranular movements, whose intensity increases more rapidly than the pressure. For the second phase theortical considerations lead one to suspect that the ratio between pressure and movement ought to follow a parabolic law. Fig. 9 contains the results of experiments made to test this. The different symbols represent separate sets of experiments, made with the same materials under similar conditions, to check each other. Parabolas have been passed through the first and the last point of each set of observations. In tests (*a*) and (*b*) the sand was simply poured into the sandbox; in (*c*) and (*d*) the sand was placed in layers, each thoroughly compacted. From the diagrams the tremendous influence of the density of the backfilling on its resistance to lateral compression is plainly apparent.

In an advanced state of the second phase the pressing body advances by jerks, which is similar to what happens during active pressure. Theoretically an approximate relation exists between the active pressure (H_2) at which the first jerk occurs and the corresponding passive pressure (H'_2), this relation being,

$$H_2 H'_2 = \tfrac{1}{2} w h^2 \qquad (3)$$

In practice H'_2 seems to be considerably greater than the theoretical value. The yielding by jerks of the retaining wall indicates that slips take place within the mass of sand. The direction of maximum tangential stress dips toward the backfilling. But the slips occur as shown in Fig. 10, simply because in the neighborhood of the surface the grains have a better opportunity to move than in the interior of the mass. Similar observations were made by Möller, who experimented with walls and backfillings resting on top of sand beds of a considerable depth.

During the second phase of active pressure, the decrease of the pressure is associated with sudden movements of the sand mass, which in turn cause spontaneous movements of the retaining wall. These events are explained as follows: The second phase starts with intergranular movements whose intensity increases more rapidly than the earth pressure decreases. If these movements existed throughout the

mass of sand there would be no opportunity for any "slip." But, as the backfilling rests on a rough surface, the intergranular movements are confined to a wedge-shaped space between the back of the wall and the remainder of the backfilling. This result was obtained by a study of the equilibrium of a mass of spheres. Along the surface of contact between stationary and moving parts of the mass the grains are forced to change their relative positions. Their equilibrium becomes more and more unstable, until they dip and overturn.

This rupture, or "slip," is simply a radical but temporary expedient to remedy a local disorder in the structure of the sand. No complete rearrangement of the grains can occur without the sand temporarily assuming within the zone of rearrangement a very loose structure (Fig. 2). The first temporary increase in bulk within the slip zone causes the first jerk. It is followed by a slight settlement within the wedge-shaped zone. Equilibrium is re-established and the intergranular movements continue until a second slip becomes necessary.

One of the conditions for the occurrence of a slip is that the stress in the sand tangential to an ideal interface (an inclined plane passing through the foot of the wall) exceeds a certain value; or, as it is expressed by the various earth-pressure theories, the value of the angle between the forces acting on the plane and the normal to the plane must exceed a certain limit. This critical value has always been identified in earth-pressure theories with the angle of repose of the material. But both theoretical investigations and experiments disprove the identification. The angle of repose of different sands ranges between 30 deg. and 35 deg., but the value of the angle of friction between wedge and plane of slip, as computed from the observed position of sliding planes, lies between the limits of 30 deg. and 54 deg. and depends chiefly on the degree of compactness of the backfilling. The angle will be called *angle of slip* and its tangent the *coefficient of slip.*

Successful attempts to determine the position of the plane of slip showed a remarkable fact. From the angle of slip the intensity of the earth pressure was computed by basing on the supposition that the plane of slip is identical with the interface of maximum tangential stress. But the computation always furnished the observed value H_2 (earth pressure at first slip), while the position of the plane of slip was determined in a state corresponding to H_3 (final breakdown of equilibrium). From this discrepancy the author concludes that *only the first slip* occurs in the direction of the maximum tangential stress, and all the succeeding slips take place along the same plane of slip. He explains this phenomenon as follows: The first slip loosens the structure of the sand within a narrow zone. Then the sand mass is no longer homogeneous, for it contains a layer of less compact material, and the following slips do not take place in the direction of the maximum tangential stress but along the plane of least resistance, i.e., along the first plane of slip.

From the facts mentioned it is evident that the slip represents only a subordinate incident in the process of lateral expansion of a backfilling. The earth pressure at first slip represents neither a maximum nor a minimum but something between. In many cases (as pressure against tunnel masonry) there is no opportunity for slip, and in these cases all theories based on the knowledge of the value of the angle of slip are worthless.

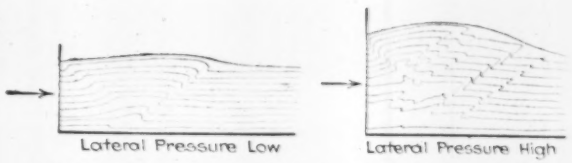


FIG. 10. DEFORMATION OF SAND BACKFILL UNDER LATERAL COMPRESSION (FORCHHEIMER)

CONCLUSIONS

The earth pressure against a perfectly rigid wall seems to be fairly independent of the density of the backfilling. For sand its value is

$$H = \tfrac{1}{2}0.42wh^2$$

Any yielding of the wall causes a decrease of the intensity of the pressure, and the minimum value, according to the density of the backfilling, corresponds to coefficients of $\frac{1}{2}wh^2$ ranging from 0.15 to even less than 0.05.

During the decrease of the pressure, two phases can be distinguished. During the first phase the relations between the intensity of the earth pressure and the distance of yield are chiefly governed by the elastic properties of the backfilling. During the second phase the influence of the structure of the sand becomes prominent.

These facts indicate the existence of relations between stress and strain in sands similar to those which are known to exist for solid bodies. "Slip," if any, occurs at an early stage of the second phase and is an event of secondary importance.

NOTES

[1] In this connection the term "slip" means the separation of the sand mass into a resting part and a sliding wedge along a well defined interface, and not the small individual slips occurring throughout the sand mass along the surface of contact of the grains. The latter ones are always "essential" events.

[2] *G. H. Darwin*, On the Horizontal Thrust of a Mass of Sand. Exc. Min. Proc. Inst. Civ. Eng. Vol. LXXI, 1883.

[3] *K. Skibinsky*, Das Gleichgewicht des Roiligen Materiales. Oesterr. Wochenschr. f.öff. Baudienst, 1916 and 1917.—Science Abstracts for 1917 mention a series of theoretical investigations of *J. Boussinesq*, Comptes Rendus 1917, also starting from the equilibrium of the individual sand grains, which shows that the necessity for returning to the elementary fact was felt throughout the world.

[4] In practice the sand grains are very small as compared with the height of the retaining wall and the preference given by the investigators to the fine sands seems to originate in an effort to adapt the experiment to the actual conditions by copying this perfectly non-essential detail.

New Zealand Harbor Improvements

Plans are well under way for the completion of wharves and sheds at Auckland, New Zealand, as well as the construction of three more wharves in order to meet the demands of the constantly increasing commerce of that city. Frequently the wharves are so badly congested that shipping must wait its turn. The Auckland Harbor Board has been authorized to place a loan of $4,866,500 for the developments indicated above, and the chairman of the board has announced that work will be pushed as rapidly as possible.—*Commerce Reports.*

Using 6-in. Bank Run Aggregate on Stevenson Dam

Large Size Gravel Dug by Steam Shovel From Nearby Pit and Made Into Concrete Chuted Into Place on Connecticut Hydro-Electric Development

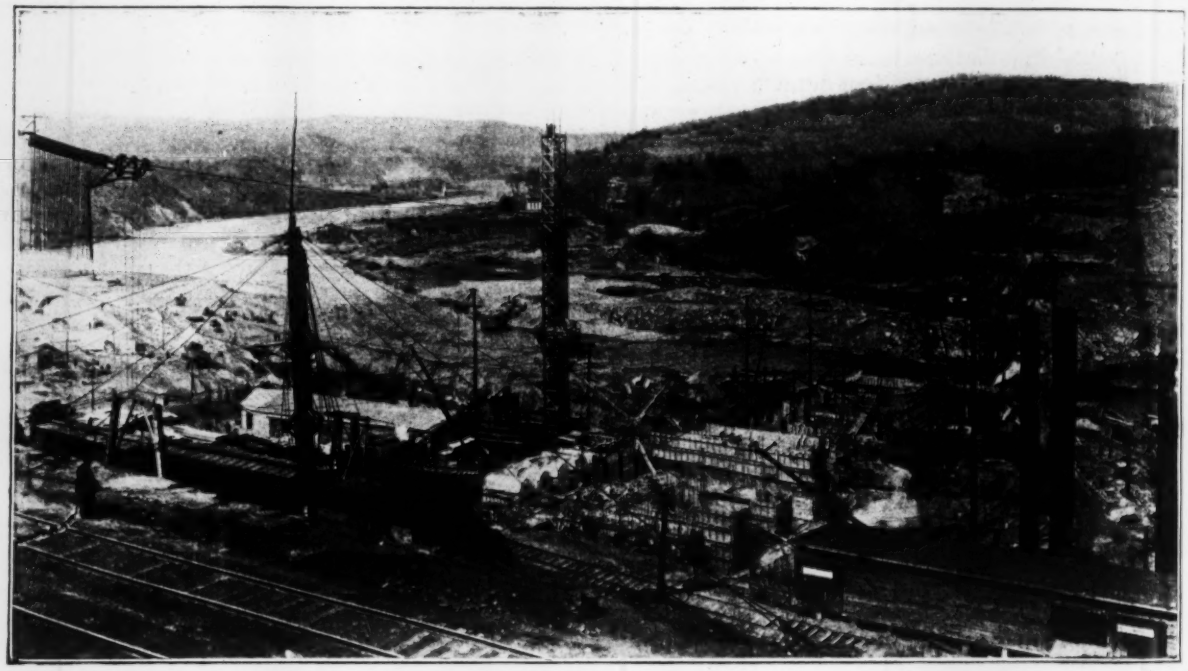
BANK RUN gravel screened down to material below 6-in. was successfully used for the concrete of the recently completed Stevenson Dam near Derby, Conn. The material was taken from a nearby deposit and economically brought to the site, where it was put through a stock concrete mixing and chuting plant, which was only slightly revised to take care of such excessive size aggregate. It is estimated that the use of the bank-run gravel saved $250,000 on the work.

The Stevenson Development of the Connecticut Light & Power Co. is located on the Housatonic River, about seven miles north of Derby, and consists of a concrete gravity section, spillway dam, concrete intake, concrete abutments and a steel and brick power house. The dam is 1,200 ft. long, about 122 ft. high, and 81 ft. wide at the base. There are about 160,000 cu.yd. of concrete in all the structures. The power house is equipped with three vertical-shaft water wheels each designed to develop 9,800 hp. at 150 r.p.m. under a 76 ft. head. These wheels are direct connected to 7,800 kva. 6,600 volt, three-phase generators. Provision has been made for the installation of a fourth unit. Current from the generators is stepped up through banks of single phase 2,500 kva. water cooled transformers to 66,000 volts for transmission to Waterbury, Conn., 26 miles away. The project also included the construction of several miles of highway, the raising of a steel highway bridge, the building of a 180-ft. span concrete cantilever bridge, the erection of several operators' dwellings, and the building of 25 miles of steel tower transmission line.

The whole area of the development is covered with an overburden of sand, gravel and boulders with bedrock at depths varying from 20 to 60 ft. This permitted a solid foundation for the dam but also suggested the possibility of the utilization of the overburden for concrete aggregate. Accordingly, an area about 1,500 ft. upstream of the dam was selected for investigation. Numerous tests of this gravel proved that an excellent quality of concrete could be made of this material using the natural bank-run and that it was necessary to vary only the quantity of cement to obtain concrete suitable for various parts of the work.

In order thoroughly to explore the gravel deposit, timbered and sheathed shafts, 6 ft. square, were sunk to groundwater level at many points over the area of the deposit. Large samples of the gravel encountered in these shafts were taken and from these samples tests were made as to the proportions of different sized aggregate. All gravel over ¼ in. mesh and under 6 in. mesh was called coarse aggregate, while all under ¼ in. mesh was termed fine aggregate. The percentage of silt was small enough to be neglected.

Other samples of the gravel taken from the shafts, passing a 6-in. mesh, were mixed with varying quantities of cement into concrete from which 6-in. and 12-in. cubes were made. In making the 6-in. cubes all aggregate over 2½ in. in diameter was removed, the reason being that larger sized aggregate would probably reduce the compressive strength of so small a cube. These cubes were stored in accordance with standards of the American Society for Testing Materials and were broken at 7 and 28-day periods. Without exception all shafts showed a uniform gravel deposit and compressive tests showed results sufficiently high to enable the engi-



LOOKING UPSTREAM FROM STEVENSON DAM UNDER CONSTRUCTION

Gravel for concrete taken from low bank bed on far side of river in middle back-ground and delivered by train to two mixing plants at foot of towers shown.

BANK WHERE 6-IN. GRAVEL FOR CONCRETE WAS DUG FOR THE STEVENSON DAM, NEAR DERBY, CONN.
Windrows shown are the rejections over 6-in. grizzly.

neers to obtain permission to use bank-run material for the concrete.

The 12-in. cubes were used to closely determine the weight of the bank-run concrete. Subsequent data showed this weight to average about 148 lb. per cu. ft.

The quantity of cement used in making the 6-in. cubes was varied in the proportions of 4½, 5½, 6½ bags to the cubic yard of resulting concrete, these amounts approximating the quantities of cement in the standard 1:3:5, 1:2½:4 and 1:2:4 mixes per cubic yard of concrete. The 4½-bag mix was used in the dam, intake, abutments and power-house foundations and the 5½-bag mix in the top 7 ft. of the dam, the bridge over the spillway and in thick walls. The 6½-bag mix was used in all reinforced work around the scroll cases, the penstocks and in very thin walls. All concrete was mixed rather sloppy wet and later proved to be of high density and exceptionally water-tight.

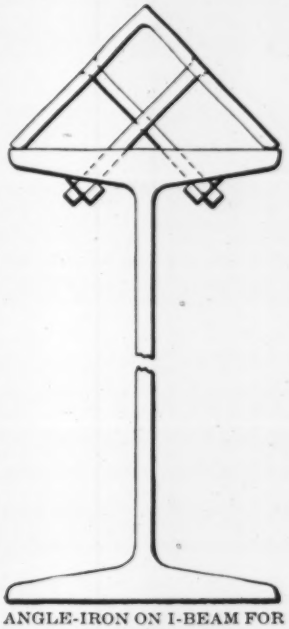
The tests proving satisfactory it was decided to take all the aggregate from the bed investigated. The overburden was stripped from the gravel deposit by ¾-yd. revolving shovels loading into bottom dump wagons. Once stripped, the concrete aggregate was loaded by a 60-ft. long boom shovel capable of cutting to a full 50-ft. bottom width and of loading at a point 26 ft. above its own track. It was equipped with a 2½-yd. dipper, which dumped onto a movable grizzly, the undersize, under 6-in. diameter, falling directly into 12-yd. dump cars. The oversize went to waste along the grizzly track, forming the windrows shown in one of the views. Grizzly bars of 60-lb. rails were first used, but this type of bar rejected an excessive amount of fine material and was soon replaced by bars made of 8-in. I-beams with angle-irons bolted to the upper flange, as shown in the sketch herewith. This bar gave a sharp upper edge and the rejections of fine aggregate were practically stopped.

Approximately 155,000 cu.yd. of aggregate were obtained in this way. This was principally hauled direct to the mixer bins and dumped directly into them. In the late winter of 1919 about 30,000 cu.yd. of aggregate was stored at a point above the high water on the right bank. This gravel formed a reserve which was used during the high-water period of that spring, during which time the gravel pit was flooded.

In certain parts of the reinforced-concrete work it was not practicable to use 6-in. aggregate because of the relatively small space between the reinforcing bars. In these cases 3-in. grizzlies were placed above the measuring hoppers and the oversize was rejected, the undersize passing directly into the measuring hoppers. This occasioned some extra work but caused the least interference with the general scheme of obtaining aggregate.

To handle the concrete two main and one subordinate mixing plants were installed, all delivering concrete through chutes and towers. Some doubt was expressed as to the ability of a chute to handle such large size aggregate successfully, but the regular 14-in. chutes of the Lakewood Engineering Co., provided however with arched braces in order to pass the large stone, were adopted and satisfactorily used.

The major portion of the concrete was mixed in two plants located one near each end of the dam. These plants were each equipped with two 1-yd. electrically driven, revolving mixers. Aggregate was brought from the gravel pit by 8-yd. cars on standard-gage track and dumped directly into bins above the mixers. From the bins the aggregate was fed by gravity to measuring hoppers, from these to the mixers, and the resulting concrete by gravity to the elevating tower buckets, where it was chuted to the various parts of the work. A small amount of concrete was mixed in a Hains gravity mixer plant located immediately below the power house on the right bank of the valley. Concrete from this plant was chuted to an auxiliary tower about midway of the length of the dam, and from there distributed into the work. The largest daily output of concrete was 1,660 cu.yd. and the largest quantity for any one month was 27,000 cu.yd., which was placed during September, 1919.



ANGLE-IRON ON I-BEAM FOR GRIZZLY BAR

It was found essential to remove all rock larger than 6 in. in diameter in order that the resulting concrete could be fully run through the 14-in. chutes. The mixers

stood up very well under the battering of the large stone. Very little trouble was experienced in handling the concrete in this manner and the engineers on the work state that, should an opportunity occur, they would endeavor to use aggregate up to 9 or 10 in. diameter providing mixers and chuting equipment could be obtained for such large sized material. Generally speaking, concrete made from aggregate containing such large material could only be used in mass work.

Daily samples of concrete were taken from each plant then running, and 6-in. compression cubes were made of these samples. These were broken at 7 and 28-day periods. Six days of the week the cubes were broken at the field laboratory and the seventh at the Mason Laboratory of the Sheffield Scientific School at Yale University. The following average results were obtained:

STEAM SHOVEL DELIVERING BANK GRAVEL FOR CONCRETE THROUGH GRIZZLY TO TRAIN

TESTS OF CONCRETE FROM STEVENSON DAM

Mason Laboratory Mix, Bags to Yd.	Time, Days	Ultimate Compr. Strength, Lb. per Sq.In.
4½	7	729
4½	28	1,576
5½	7	860
5½	28	1,909
6½	7	1,147
6½	28	2,133
Stevenson Laboratory		
4½	..	589
4½	..	1,171
5½	..	770
5½	..	1,292
6½	7	941
6½	28	1,555

It is noticeable that the results obtained in the Mason Laboratory were higher than in the field laboratory. This is probably due to the difference in character of the two machines used in testing the cubes, the one in the Mason Laboratory being an expensive well-designed machine, while in the field an inexpensive hand pump operated machine was used.

The development was constructed for the Connecticut Light & Power Co., of Waterbury, Conn., by the J. A. P. Crisfield Contracting Co., Philadelphia, contractors, with C. W. Blakeslee & Sons, of New Haven, sub-contractors. The design was furnished by the J. A. P. Crisfield Contracting Co. and the Birkinbine Engineering Offices, Philadelphia. Construction work was done under the direct supervision of H. J. Hoard, chief engineer, and E. H. Burroughs, assistant chief engineer, of the J. A. P. Crisfield Contracting Co. These two engineers furnished the material from which this article was prepared.

Ocean Sewage Disposal in Relation to Bathing Beaches

By Lynn Perry
Assistant Professor of Civil Engineering, Lafayette College, Easton, Pa.

DISPOSAL of sewage into the ocean or into watercourses in close proximity to bathing beaches is frequently considered prejudicial to the health of the bathers. Whether or not this practice actually pollutes the water bathed in, or not, the esthetic sense of the public usually demands that some other place of disposal be utilized, even at considerable additional public expense. Some seashore municipalities have recently incurred a heavy expense by constructing tanks, filters and disinfecting plants to purify their sewage so it may be disposed of into estuaries where shellfish are grown, rather than prejudice the purity of their bathing beach water.

The public health laws of New Jersey do not permit a municipality to dispose of its sewage into any watercourse within the borders of the state or into the ocean except in accordance with plans approved by the State Department of Health. That Department had uniformly required municipalities desiring to dispose of their sewage into the ocean to provide a minimum treatment of sedimentation and an outfall pipe sufficiently long to conduct the tank effluent into the ocean beyond the surf. A section of such a sewage tank and outfall is shown in Fig. 1. The effect of this method of sewage disposal on near-by bathing beaches is the subject of this article.

In the summer of 1916 the State Department of Health of New Jersey conducted a series of investigations with a view of determining what effect, if any, disposing of sewage into the ocean has upon the quality of water at bathing beaches. The cities at which investigations were conducted were carefully selected as being both large and representative seaside resorts.

FIG. 1. SECTION THROUGH SEWAGE TANK AND OCEAN OUTFALL

During the period of investigation, the wind was south, southeast, and northeast and low tide occurred near midday, giving ebb tide in the forenoon and flood tide in the afternoon. The direction and velocity of the effluent after being discharged into the ocean was determined by the use of two surface and two submerged floats. The surface float (Fig. 2) was constructed to submerge to a depth of 9 in. No portion of the float protuded above the water surface. The float was not affected by the wind, undertow, or under-currents of the ocean except in the same measure as the surface film of water was affected. Each time the surface

float reached the breaker line, it was removed from the water to prevent its being washed ashore by the surf. The location of no samples was governed by the action of the surface float. The submerged float was of the usual wood type, 1½ x 1½ in. in section, 5 ft. long, with a ½-in. spar projecting from the surface end. The projecting spar was so small that the wind effect was very small but it was necessary for purposes of observation. The floats were placed in the ocean over the outlet and followed by observers and sample collectors in a boat. Fig. 3 shows in dotted lines the courses traveled by the surface float and in solid lines the courses followed by the submerged float.

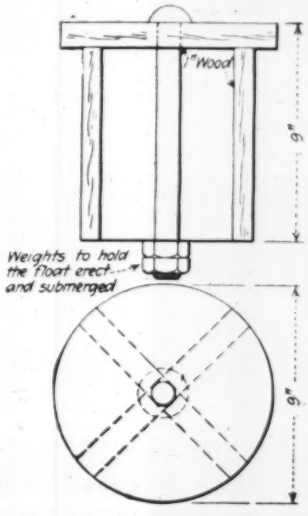


FIG. 2. SURFACE FLOAT FOR TRACING SEWAGE FLOW

The samples may be divided into two classes: Those collected from the boat that followed the float or "float samples", and those collected by wading out from the shore, or "surf samples." All samples were collected with care about 6 in. under the water surface. It was at first the intention to collect samples at hourly intervals, the "float samples" from the boat near the float and the "surf samples" at predetermined points along the beach. The first day's experience, however, showed that the float traveled so rapidly that it arrived in front of a bathing beach of another municipality before the hour had elapsed, thus samples collected at hourly intervals were so far apart and the tank effluent so highly diluted with ocean water that they showed a very low bacterial count. This was true even of the sample collected one hour from the outfall. Accordingly, on the second day the time interval of collections was reduced to 30 minutes, and on the third day, further reduced to 15 minutes. The "float samples" were not kept over two hours before plating and the "surf samples" not over half an hour.

Fermentation tubes with lactose bile medium were used for presumptive tests and lactose agar stained with azolitmin was used for plate culture medium. Both tubes and plates incubated at 37 C., using a portable field incubator.

The "surf samples" checked very closely. The results show: (1) Immediately ashore and for some distance on each side of the outfall no serious pollution exists (and no bathing beaches are located at these places). Indeed, it is doubtful if any pollution exists due to the outfall. (2) At, and in close proximity to, the bathing beaches on both sides of the outfall, both the total and red colony count increased rapidly, which, in view of the results from the float samples, was probably local and may be attributed to the bathers.

When the float was carefully placed over the center of the outfall and travelled near the center of sewage flow, the bacteria counts were remarkably consistent. But when the float travelled near the edge of the main flow, low bacteria counts resulted from the start, the samples being collected on the edge of and not in the center of the horizontal flow of the tank effluent. Samples collected equidistant from the outfall, in the center line of flow, substantially check each other. The results are shown on Fig. 3.

The results indicate: (1) That the tank effluent travels approximately parallel to the shore. (2) That the tank effluent travels in a very narrow stream which is probably not over 25 ft. wide at the outfall. (3) That the enormous quantity of ocean water rapidly dilutes or oxidizes the tank effluent. (4) That the surface film of water ultimately reaches the surf but only after it has become so diluted that it is of a higher bacterial standard than surf water where the bathers are active.

During two days of heavy northeast storm the surf was so high that it was too dangerous to launch and land the boat. There were no bathers and only "surf samples" were collected. The results from the samples were not as reliable nor as consistent as the others. They indicate that, during extreme northeast storms, after over 24 hours duration, highly diluted effluent begins to reach the shore southwest of the outlet.

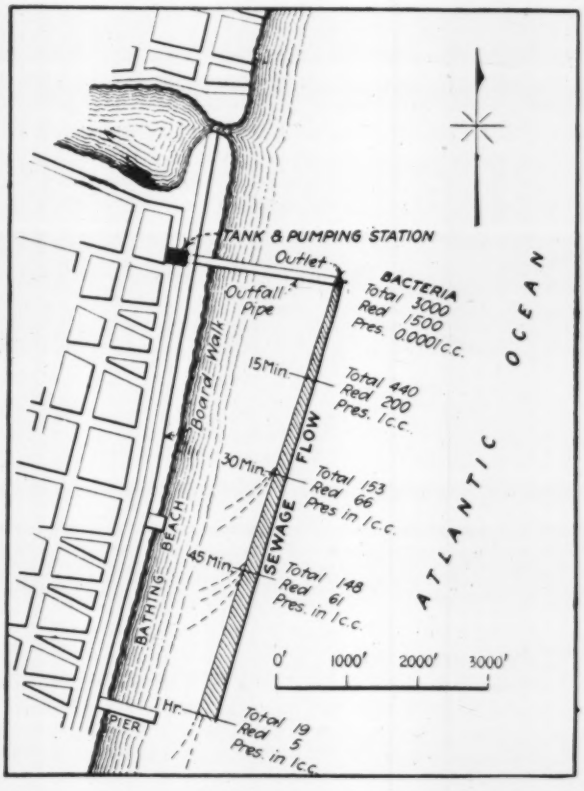


FIG. 3. COURSE OF SEWAGE AND BACTERIAL COUNTS
Averages of bacterial counts at 15-minute intervals.

In conclusion, these investigations indicate that thoroughly settled sewage, when discharged into the ocean well outside of the surf, is not likely to impair the bacterial quality of water at bathing beaches. The investigations were planned by Chester G. Wigley, then chief engineer, and the field work was executed by the writer, then sanitary engineer, and George D. Norcum, chemist of the New Jersey State Department of Health at Trenton.

Immigration and the Construction Industry

Labor Shortage Not Yet Relieved by Growing Number of Immigrants Who Are Mostly Women and Children—Prohibition Said To Be No Deterrent

ENGINEER and contractor alike have been confronted of late with two questions, entirely new, and upon which opinion appears to be about evenly divided: Is prohibition having and adverse effect upon immigration, and is any relief from the labor shortage in sight from foreign sources? Studies of reports of the U. S. Immigration Commissioner, the U. S. Bureau of Labor Statistics, and the U. S. Geological Survey, and information coming from the Superintendent of Ellis Island, New York, seem to point to a negative answer to the first question and a positive answer to the second.

The contention has often been made and as often contradicted that prohibition is abetting the labor shortage through immigration restriction, and that, were the United States to allow the manufacture and sale of light wines and beer, a certain class of immigrants, from which the construction industry draws its greatest per cent of common labor, would again be attracted to this country. Information from immigration officials and analyses of immigration figures and reports are our best sources of information upon this question.

Prohibition No Bar

In the opinion of P. A. Baker, superintendent of Ellis Island, a stronger bar than prohibition will have to be set up before the foreigners, no matter of what race or color, will be kept out of this country. It is his assertion that before the war approximately one man in one hundred immigrated here with the intention of becoming a citizen. The vast majority come because they are attracted by high wages, plenty of work and the picture of soon returning, affluent, to their native countries. With this condition prevailing the immigrant, believes Mr. Baker, is going to be little concerned with the 18th Amendment to the Constitution. Reports on immigration for the past year and a half, and particularly for the first six months of 1920, point to a substantiation of Mr. Baker's opinion.

No figures are yet available for 1919 upon immigration for the entire United States, but figures are available for the Port of New York, which, in normal years, handles approximately three-quarters of the total immigration. However, this percentage has varied considerably during the last four or five years, and just what the present per cent of total immigration handled by New York is, cannot be known until the figures are correlated. During 1919 it was frequently said that a great many more emigrants were departing than immigrants were arriving, a contention upheld by figures contained in Table I, compiled from information secured from Ellis Island officials. Although approximately 75 per cent more people returned to European countries than arrived during 1919, it must be remembered that people of European birth were given the opportunity of visiting their war-ridden countries for the first time in five years.

TABLE I—ALIEN ARRIVALS AND DEPARTURES PORT OF NEW YORK, FOR CALENDAR YEAR 1919.

Mo.	Arrivals	Departures	Mo.	Arrivals	Departures
Jan.	2,589	9,211	July	9,432	28,354
Feb.	2,975	9,571	Aug.	9,430	32,901
Mar.	5,850	17,201	Sept.	20,577	29,746
Apr.	11,355	15,946	Oct.	24,641	24,833
May	7,462	17,786	Nov.	17,557	36,459
June	9,485	27,998	Dec.	34,529	24,549
			Total	155 882	274,455

Emigration Not New

The emigration of so many people from the United States is believed by many to be a thing of recent origin, or an after-the-war condition. It is not at all new, and a glance at Table II, giving the total alien immigrants arriving during the years named, and the alien emigrants departing, shows that there has always been a heavy outward movement during pre-war normal years amounting to from 40 to 50 per cent of the total immigration. Theh figures, given in Table II, do not include any American citizens travelling abroad, but cover only alien emigrants. Since the beginning of the war this percentage increased, until during 1919 a great many more emigrants left the United States than immigrants arrived, as Table I shows.

TABLE II—COMPARISON OF EMIGRATION AND IMMIGRATION FOR ENTIRE COUNTRY 1907-1918.

Year	Immigration	Emigration	Year	Immigration	Emigration
1907	1,285,349	569,882	1913	1,427,227	611,924
1908	782,870	714,828	1914	1,403,081	633,805
1909	751,786	284,800	1915	434,244	384,174
1910	1,041,570	380,418	1916	366,748	240,807
1911	1,030,300	518,215	1917	362,877	146,379
1912	1,017,155	615,292	1918	211,853	193,268

Despite the fact that emigration overbalanced immigration during 1919, the exact opposite is promised if the figures available for the first six months of 1920 can be used as a basis. Recent reports from Ellis Island are to the effect that from January to June, inclusive, of this year, 197,956 immigrants arrived at Ellis Island and 152,537 emigrants departed. This shows a favorable balance of immigration over emigration of approximately 25 per cent.

As perviously stated, the Port of New York handles approximately 75 per cent of the immigration during normal years. From 1909 to 1914, inclusive, this percentage ranged between a maximum of 77 for 1909 and a minimum of 72 recorded in 1912 and 1914. However, since the beginning of the war and particularly since the entrance of the United States into it, the Port of New York has been more or less closed to immigration and emigration so that this percentage has decreased perceptibly. The percentage handled by the port of New York was, according to the reports of the Immigration Commissioner, for 1915, 55 per cent; 1916, 47 per cent; 1917, 44 per cent; 1918, 26 per cent, and 1919, 19 per cent. Thus it will be noted that not only has New York so far this calendar year handled approximately the same number of alien immigrants as came into the country during 1918, but that it will probably report the arrival of approximately 400,000 immigrants for the entire year.

It is undoubtedly a false premise to assume that during 1920 New York will handle 75 per cent of the total immigration, for considerable quickening has been given

shipping in all other American ports since the war. However, assuming that the 400,000 immigrants represent 75 per cent of the immigration for the year, it will give us a total immigration of 525,000 for 1920. If, on the other hand, that 400,000 is but 19 per cent, New York's contribution in 1919, a maximum of 2,000,000 for the whole United States may be a possibility. As such a figure seems improbable, it is better to strike an average, 750,000 immigrants appearing to be a reasonable one.

Though it is apparent from the foregoing figures that prohibition has little effect upon immigration, it is necessary to go further and find out, if possible, if among these immigrants the normal percentages of common labor and skilled mechanics are numbered. As no segregation of immigrants is made at Ellis Island, all figures of that sort being compiled at Washington, no accurate figures are available now. However, Superintendent Baker asserts that, although incoming ships have capacity lists, both in cabin and steerage, the great majority of immigrants arriving are aged couples, women and children, and Italian reservists. According to him very few male adult laborers are counted among the arriving immigrants. While it is not ascertainable what effect the war had upon the quality of immigrants it is obvious that regulations against emigration of adult laborers from certain European countries have set up a considerable barrier against the United States receiving the quality of immigration it did in pre-war years.

In trying to discover what the labor shortage may be, or how it can be relieved through immigration, the presentation of certain data may clarify the situation. From 1907 to 1914, inclusive, the reports of the Immigration Commissioner show that an average of 27 per cent of all immigrants reported no occupation. The reports also show that of the total immigrants admitted an average of 19 per cent class themselves as common laborers. This figure is secured by averaging the yearly percentages for the years 1907-1914, inclusive, percentages which range from a minimum of 16.2 per cent to a maximum of 23.2 per cent. It is also noted that during these same years the average annual percentage of immigrants in the building trades was 3.3 per cent ranging from a minimum of 3 per cent in 1909 to a maximum of 4 per cent for 1907 and 1911. These figures are reproduced in detail in Table III. From such figures

TABLE III—NUMBER IMMIGRANTS IN BUILDING TRADES ADMITTED DURING PERIOD 1907-1914, INCLUSIVE

	1907	1908	1909	1910	1911	1912	1913	1914
Carpenters	20,656	11,394	8,606	13,887	13,172	11,034	15,035	15,755
Engineers	1,464	1,438	826	1,695	1,638	1,331	1,594	1,684
Iron and Steel Workers	2,249	1,652	1,354	2,845	2,544	1,366	1,728	1,637
Machinists	2,224	2,045	1,831	3,336	2,902	2,098	2,725	2,676
Masons	11,308	4,589	3,860	6,257	5,728	4,555	7,377	7,772
Mechanics	1,745	1,325	644	1,449	1,536	1,342	1,853	1,954
Metal Workers	960	715	444	808	850	669	793	791
Painters	5,239	3,254	2,086	3,460	3,667	2,816	3,888	4,178
Plasterers	1,013	365	477	654	454	319	416	425
Plumbers	906	621	473	745	833	584	778	761
Stonecutters	2,052	1,166	1,337	1,491	1,204	972	1,111	1,036
Tinners	1,635	960	436	720	825	737	879	1,047
Totals	51,451	29,524	22,374	37,347	35,353	27,823	38,179	39,666
Per cent Immigration in Building Trades	4%	3.8%	3%	3.6%	4%	3.3%	3.2%	3.3%

it is seen that in pre-war years approximately the same character of immigrants were received from year to year. Comparing the character of immigration during 1918 with the pre-war years, it is found that 56.5 per cent of the total immigration reported no occupation. In the building trades, however, the same percentage was maintained, 3.2 per cent of the 1918 immigration being numbered in the building trades. Common labor fell off somewhat, only 13.3 per cent being classed as common labor.

Three facts are then noticeable in the foregoing review and comparison: (1) That immigration for the calendar year 1920 promises to eclipse any year since 1914; (2) that the percentage of immigrants counted in the building trades remains the same, and (3) that common labor has undergone an appreciable decrease.

FORECAST OF CONDITIONS

Besides the promise of a greatly increased immigration this year, analyses of available figures lead to various forecasts of conditions. What is evolved in the following paragraphs is of little value for present application, but it may be of value to furnish a basis for predicting the status of the construction industry as affected by immigration.

In the census of 1910 it is shown that there were in the building trades, excluding common labor, though including apprentices, 2,409,348 persons. In the same year the U. S. Geological Survey reported that 51 of the larger American cities had issued building permits aggregating in value $734,112,998. As little unemployment was noted in that year it can be taken as a normal one. Therefore, one can assume that all men in the building trades were actually employed that year. Although the men in the building trades, as reported in the 1910 census, were not all employed in the 51 named cities, apparently the cities are a fairly accurate barometer of building conditions throughout the United States at that time. It is possible to make a ratio between the number of men employed in the building trades in 1910 and the value of building construction reported by the 51 cities. This shows that in 1910 a building trades workman had a construction value of $305.

TABLE IV—COMPUTATION OF MEN IN BUILDING TRADES WITH KNOWN 1910 CENSUS DATA AS BASIS

	—Building Permits in 51 Cities—			
Year (1)	Reported Value (2)	Corrected Value (3)	Men in Bldg. Trades (4)	Number Needed (5)
1910	$734,112,998	$734,112,998	2,409,348	2,409,348
1911	687,506,961	687,506,961	2,446,695	2,250,000
1912	738,989,710	738,989,710	2,482,045	2,420,000
1913	673,220,625	673,220,625	2,509,868	2,200,000
1914	619,752,354	619,752,354	2,548,047	2,025,000
1915	641,769,199	625,000,000	2,587,713	2,050,000
1916	780,183,970	710,000,000	2,598,513	2,330,000
1917	512,576,744	394,000,000	2,608,413	1,290,000
1918	302,571,607	197,000,000	2,618,113	645,000

Maintaining this figure throughout the years, starting with 1910 as a base, the figures are tabulated in Table IV. This gives the permit values of the same 51 cities for 1910 to 1918, inclusive, and a column for corrected values, the latter being values with the changes in the value of a dollar taken into account. To convert from reported to corrected value, a mean between the index numbers given by the Bureau of Labor Statistics in its report upon the cost of 22 articles of food, and the index numbers given by the National Industrial Conference Board, is taken. Column four in Table IV begins with the number of men in the building trades as enumerated in the 1910 census. To that figure is added for the years 1911, 1912, 1913 and 1914 the actual number of immigrants listed in the building trades according to the figures of the Bureau of Immigration. From 1915 to 1919, inclusive, the percentage of 3.3 of the total immigration for that particular year is added. No account is taken of the number of men already citizens who enter the building trades. Dividing then the values of build-

ing construction, listed in column three, by the construction value of one man—$305—column five is made up.

The number of men enumerated in the building trades according to column four, as compared with the number of men actually needed to do the work reported by the the 51 cities, shows a considerable discrepancy, particularly beginning with the year 1913. However, it will be remembered that a period of unemployment began about that time and during the fall of 1914 and the spring of 1915, various surveys were made to determine, if possible the amount of unemployment in the various trades. Of these surveys the Bureau of Labor Statistics made perhaps the most authentic and exhaustive one. In its review of unemployment it was announced that an average of 20 per cent of all men in the building trades were unemployed during the latter part of 1914 and early in 1915. This average is taken by averaging the percentages reported from the following cities: Boston, Chicago, Philadelphia, Pittsburgh, St. Louis, Bridgeport, Cleveland, Duluth, Kansas City, Louisville, Milwaukee, Minneapolis, St. Paul, Springfield (Mo.), Toledo and Wilkesbarre. The minimum of these cities was 12.2 per cent and the maximum 27.7 per cent, the average being 20 per cent.

Unemployment Discount

Doubtless some unemployment existed during 1913, but as the survey was for 1914 and 1915 only, we can apply the percentages of unemployment during just those two years. If it be considered, therefore, that during 1914 and 1915 20 per cent of all men in the building trades were unemployed, we find that number to be approximately 500,000 in each year, the amount of the discrepancy between columns four and five.

This is as far as the analysis holds, because during succeeding years war disturbed the normal trend of things. Building construction cannot during the war be taken as representing the same percentage of total amount of money involved in construction work as before the war. In normal years this ratio of value of building permits in the 51 cities to the total amount of money in contracts awarded in the district north of the Ohio River and east of the Mississippi River was around 60 per cent.

While it is possible to make a very rough approximation of the shortage of building mechanics, using the data in column four, Table IV, and the various agency reports as a background, it is believed that the elements entering into even a rough approximation are so uncertain that it would be of little value. In the first place, if such a course be followed, several things are first to be considered. It is known that during the war the value of building permits reported by the 51 cities was far below a normal percent of the total amount of money involved in contracts. Then in the figures shown in Table IV no account is taken either of the adjustments made in wages and hours of labor, which must of necessity increase the number of laborers to do a particular job, nor of the reputed decrease in the efficiency of labor. Then the war induced many men in the builidng trades, drafted into service, to change their trades after being discharged. A tremendous impetus has been given industrial construction within the past few years, and this fact would no doubt materially decrease the percentage of total contracts awarded that the building permits recorded by the 51 cities represent.

Though of little present value, Table IV may serve students of the construction situation with some information upon which an approximation of labor shortages may be based in the future.

Recent Developments in the Manufacture of Cast-Iron Pipe

Abstract of Paper Read Before the New England Water Works Association, September, 1920.

SOME tentative new methods in the manufacture of cast-iron pipe were briefly mentioned in the report of the committee on standard specifications for cast-iron pipe, submitted to the recent convention of the New England Water Works Association by Frank McInnes, chairman, Boston, Mass. These methods were considered primarily in their possible relation to the question whether existing standard specifications shall be changed to require uniform outside diameter of pipe. The committee said:

One of these developments, the use of new methods of core making, where the thickness of material upon the core bar is reduced to a minimum, makes a uniform outside diameter less feasible than with the older methods. On the other hand, another company is producing a high tensile iron developed by high temperature on electric furnace and with this iron the range in thickness for different classes of pipe is so much reduced that the difficulty of obtaining a uniform outside diameter without too great multiplication of core bars, is, in great part, removed.

In one of the foundries of the United States Cast Iron Pipe & Foundry Co. the use of core bars was observed. The method is claimed to produce uniformly smooth inside walls and has been adopted for this reason. It is in line with the manufacturers' contention that with the greater thickness of material on the core bar, necessarily incidental to a uniform outside diameter, it will be difficult to make pipe with smooth interior surfaces. The product as observed was noticeably clean and smooth.

In the foundry of the American Cast Iron Pipe Co. iron with a tensile strength of 30,000 to 40,000 lb. per square inch was being used—made possible by high temperature treatment in an electric furnace of a metal partly taken direct from a blast furnace and partly from the foundry cupola.

The feasibility of using iron of greater tensile strength than in the past is a matter of general interest. The old line manufacturers say they have been all through this question and are convinced that high tensile iron and the resulting thinner pipe cannot be made a success. Such pipe are, however, being sold and, in this age of advancing freight rates, the possibility of reducing weight is becoming an important consideration.

Another interesting development in the making of pipe is the De Lavaud Centrifugal process in use at the plant of the National Iron Corporation, Toronto. By this method the iron is poured into a horizontal mould revolving at high speed which centrifuges the metal against the mould to the desired thickness. The pipe so made is said to have a tensile strength of 35,000 to 39,000 lb. per square inch. Obviously this method particularly lends itself to a uniform outside diameter.

Current American Shipbuilding

On Aug. 1, 1920, private American shipyards were building or under contract to build for private shipowners 389 steel vessels of 1,335,721 gross tons, compared with 334 vessels of 1,306,956 gross tons on July 1, 1920, according to a recent issue of *Commerce Reports*. These figures do not include Government ships building, or contracted for by the United States Shipping Board out of money voted by Congress.

Progress Toward Relief from Water Shortage at Salt Lake City

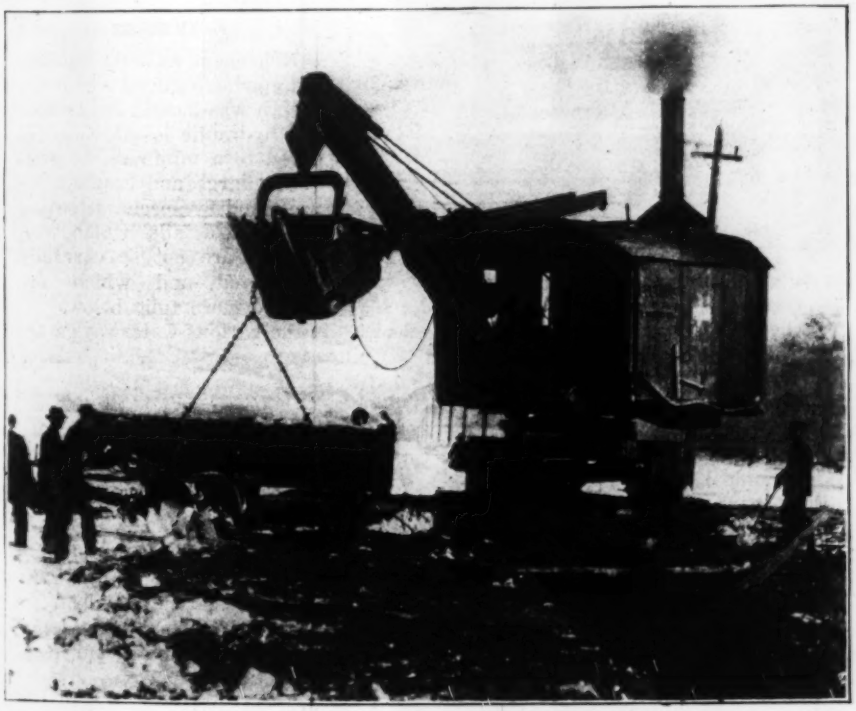
BY SEPTEMBER Salt Lake City will be protected by the addition of a 40,000,000-gal. daily high-line conduit against a water shortage which has seriously restricted consumption during the past few irrigation seasons. The existing inadequacy of supply mains has become acute because of high per capita consumption and the growth of the city toward the eastern city limits requiring a supply at a high elevation. The average consumption in this semi-arid intermountain city is 203 gal. per capita, with a winter average of 160 gal. and a summer average of 250 gal. However, for one or two months the average consumption reaches 215 gal. and the peak load, lasting for 1 or 2 days, reaches 350 gal. per person daily.

During the early summer the supply comes mainly from City Creek which flows down a canyon immediately above the city. As the summer advances the City Creek flow diminishes materially and the supply must come largely from Big Cottonwood and Parley's creeks, mountain streams emerging from the mountains south and east of Salt Lake. It is to supplement the carrying capacity of the existing mains from these two supplies that the three miles of 48-in. reinforced-concrete conduit is now being rushed to completion from the mouth of Parley's canyon on a 0.15 per cent grade along the foot hill contours to a high point in the city distribution system. At the upper end the new conduit connects with an existing line by an 800-ft. inverted siphon of 42-in. riveted-steel pipe under a maximum head of 185 ft. At the lower end 1½ miles of 30-in. cast-iron pipe leads the water into the distribution system at several points.

Excavation for the cast-iron section in town as well as the laying of the pipe is being done by steam shovel. The concrete pipe is made at a point near the center of the conduit line, the pipe being transported to the trench by a two-wheeled cart hauled by a two-horse team.

The contractor, P. J. Moran, is laying from 275 to 300 ft. of pipe per day of eight hours. The total cost will be $385,000. Some of the unit prices bid are as follows: Earth, excavation and backfilling, $1.60 per yard; solid rock, excavation and backfill, $5; 42-in. steel pipe, laid, $15 per foot; 42-in. concrete pipe, laid, $6.25; 30-in. class A, cast-iron pipe, $13.90 per foot, 30-in. Class B, cast-iron pipe, $15.30.

All engineering and supervision are being handled under the direction of S. Q. Cannon, city engineer.



LAYING 30-IN. CAST-IRON PIPE WITH STEAM SHOVEL.

State of Missouri Plans 6,000-Mi. Highway System

PLANS are being made for the establishment of a 6,000-mi. system of state highways in Missouri, as provided by the McCullough-Morgan amendment to the Hawes road law which went into effect in March, 1919, and which required that this system be designated by the state highway engineer (subject to the approval of the state highway department), and be surveyed within three years.

This law provides that the roads are to be distributed among the 114 counties of the state in proportion to their areas and mileage of county roads. Due regard is to be given to directness and continuity of routes, low grades, economy of construction and maintenance and the needs of the people of the country, but no county is to have less than 50 mi. of state roads.

State or Federal aid to the extent of $1,200 per mile is made available for the purpose of road improvement, and of constructing culverts and bridges of 20-ft. span and under. Where the cost of grading and draining a road exceeds $1,200 per mile, or when a better road is desired, the law allows the additional amount to be shared equally by the local authorities and the Federal Government. Where the cost of grading and construction of culverts and bridges exceeds $1,200 per mile but the local authorities are unable or refuse to provide the necessary additional money, the law allows the state highway department to concentrate money due for these roads and to construct such mileage as is deemed advisable. Up to August, 1920, surveys had been made for over 4,000 mi., and complete plan submitted for 1,730 mi.

Design of the 37,500-Hp. Turbines at Niagara

Hydraulic Features of the 100,000-Hp. Addition to the Hydro-Electric Power Plants on the American Side of the Great Falls—Designs of the Two Types of Turbines

The general features of the 100,000-hp. addition to the plant of the Niagara Falls Power Co. were given in an article in "Engineering News-Record," Sept. 23, 1920. In the following article the hydraulic design of the plant is treated by George R. Shepard and N. R. Gibson, engineers of the company, and W. M. White and Lewis F. Moody, engineers, respectively, for the two companies which designed and built the huge turbines, at this time the largest in the world.

Hydraulic Problems of the Design

By George R. Shepard and Norman R. Gibson
Assistant Chief Engineer Hydraulic Engineer
The Niagara Falls Power Co., Niagara Falls, N. Y.

EXPERIENCE of the power company in former developments at Niagara led to the adoption, in 1906, of the horizontal shaft units for the plant then being designed. However, in 1916, when most of the preliminary engineering on the extension to Station No. 3 was undertaken, the Kingsbury and General Electric thrust bearings had reached a stage of perfection that eliminated the engineering problem in connection with the supporting of the weight of the rotating parts of the unit. These mechanical difficulties having been eliminated, the efficiency of the unit became the deciding factor, and vertical units were used as offering the best opportunity for the development of a high efficiency. The surface elevation of the water in the lower river fluctuates through a vertical range of about 30 ft., so that the vertical unit offered the only means of getting the waterwheel runner low enough to get the full benefit of the low-water stages, and at the same time have the electrical machinery high enough to be safe from damage during high-water stages. The available head obtained by this arrangement varied from 210 to 220 ft., the mean being 215 ft.

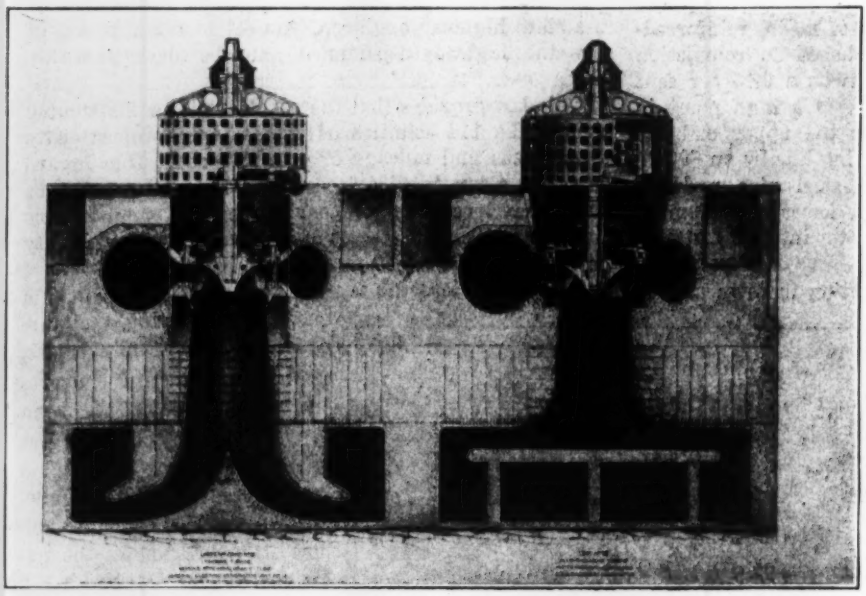


FIG. 1. SECTIONS THROUGH THE NEW NIAGARA TURBINES

Having determined the head, the maximum safe specific speed for the Francis type of runner was at once known, and gave a rough idea of the limitations of speed and load for each unit. There was available for development 4,400 sec.-ft., and the problem was to determine the number and size of units which would most efficiently develop that amount of water. Consideration of this feature and the rough limitations set by the specific speed, led to the selection of three units as the proper number to use. The fixed number of units led directly to the point of maximum efficiency for each single unit, which was set at the point of use of 1,500 sec.-ft. The experience of the waterwheel builders was that in order to arrive at this result it would be necessary to design the wheels for a maximum output of from 10 to 12 per cent in excess of the point of maximum efficiency. The builders further agreed that with the hydraulic efficiency naturally to be expected, this maximum output would be 37,500 hp. The combination of this maximum output with the specific speed and the periodicity of the current to be generated fixed the speed of the unit at 150 r.p.m.

Hydraulic Design and Efficiency

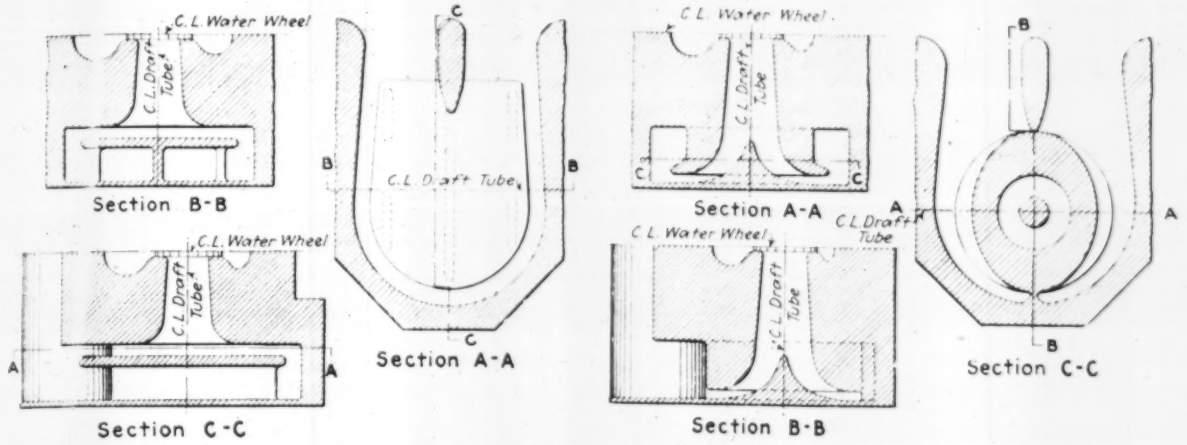
Efficiency was the underlying principle upon which the design was based, and to this end hydraulic losses were reduced to a minimum by providing large and easy waterways and by special attention to some details which frequently are not so carefully considered and which are treated more fully below.

The nominal velocities in the various waterways were as follows:

Canal	4.5 ft. per sec.
Forebay	1.5 ft. per sec.
Through tracks	1.8 ft. per sec.
Penstock entrances	3.0 ft. per sec.
Penstocks	8.0 ft. per sec.
Entrances to turbine gates	17.3 ft. per sec.
Top of draft tube	20.0 ft. per sec.
Exit from draft tube	5.0 ft. per sec.
Tailrace	3.5 to 4.5 ft. per sec.

The resulting hydraulic losses from head water to tail water, exclusive of turbine efficiency, has been found to be approximately 9.65 ft. at normal load, or 0.3 per cent of the total average head available. The net hydraulic efficiency of the plant, including turbines, is 92.5 per cent.

Having determined the over-all characteristics of the installation, the first minor problem to obtain consideration was that of the proper shape of the draft tube.



DETAILS OF WHITE'S HYDRAUCONE REGAINER (ON LEFT) AND (ON RIGHT) MOODY'S SPREADING DRAFT TUBE

The company's engineers had done considerable pioneer work along this line for a period of some twenty years, but neither their experience nor the literature on the subject appeared to point to the proper solution of the problem. The function of the draft tube is twofold; one, to utilize the static head between the bottom of the waterwheel runner and the surface elevation of the tailrace, and the other to regain as much of the discharge velocity head as is possible. Preliminary designs of the runner indicated a discharge velocity of between 18 and 20 ft. per sec., which represents a substantial part of the entire head.

Two Kinds of Draft Tube

The ideal draft tube would be a straight tube of uniformly enlarging cross-section, the expansion angle being in the neighborhood of 4 deg. This ideal shape is impracticable in most installations on account of the prohibitive cost of the excavation required. It was necessary, therefore, to determine some form of tube which would accomplish the same purpose in a much shorter vertical distance.

In collaboration with the designers of hydraulic turbines, considerable attention was given to the design of draft tubes, and after experimenting with about twenty-five draft-tube models of various designs, two types were finally adopted, and constructed in the power-house foundations. These are shown in Fig. 2, one being the hydraucone invented and patented by W. M. White, of Milwaukee, and the other the spreading draft tube patented by Lewis F. Moody, of Philadelphia. The efficiencies of these two tubes, as shown by Fig. 3, were practically the same, and both gave results which were far in excess of the efficiencies of any other model of equal dimensions.

The effect of these draft tubes is to maintain flow parallel to or radially from a central axis, with a gradual diminution of velocity until the velocity of discharge is finally reached. Such designs allow free play for the whirls in the water as it leaves the runner until the velocity of whirl has been greatly reduced. Large losses which occur when the direction of flow changes are thus prevented. The passages are also designed so that the hydraulic friction losses throughout the tube are reduced to a minimum.

As far as can be ascertained, the adoption of these draft tubes has been fully justified by the results attained, such as a direct gain in efficiency and the freedom from excessive vibration of the machinery.

The efficiencies of the hydraulic turbines were given particular consideration, but no guarantees were exacted from the manufacturers. Instead, each manufacturer agreed to furnish a turbine which would have the highest efficiency that it was possible to attain under the existing state of the art of turbine building. With this ideal before them, it is certain that each builder has put into his work the best design, material and workmanship of which he was capable.

New Method of Testing Wheels

Provision for testing these wheels in place was made by building a testing chamber so that pitot tubes and other equipment might be readily installed. It is expected that tests of the wheels may be made by using pitot tubes and chemical gagings, but up to the present, for various reasons, it has not been possible to have such tests carried out. Complete efficiency tests have been made, however, by a new indirect method recently invented by N. R. Gibson. This method, sometimes called the "pressure-time process" utilizes the principle governing the relation between velocity destroyed and rise of pressure, and by means of specially designed apparatus, patents for which have been applied for, there is produced a diagram, called the pressure-time diagram, from which the mean velocity in the conduit may be calculated. The apparatus is attached to a small piezometer tube tapped into the wall of the penstock at any convenient point, and records with respect to time, on a sensitized film or paper, the changes of pressure that occur in the penstock. The changes of pressure are produced by the simple process of closing the turbine gates, or, in this case, the Johnson valve.

The procedure for a test by this method may be briefly described as follows: The turbine gates are put on hand control, and steady conditions of load on the unit are maintained for several minutes until the flow in the penstock has become as uniform as possible. Readings of headwater and tailwater elevations are taken in the usual manner, and the pressure head at the entrance to the turbine gates is observed by gage or piezometer, so that allowances may be made for the loss of head in the penstock. Measurements of the generator output are taken by calibrated watt-

meter and auxiliary instruments, and, if the unit is separately excited, ammeter and voltmeter readings of the exciting current are also obtained. When conditions have been steady for several minutes, the turbine gates are closed gradually by operating the hand-control pilot valve. During the closure the load thrown off the generator is taken up by the other units operating

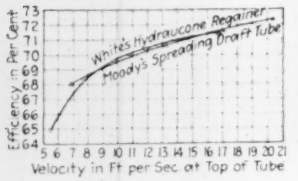


FIG. 3. EFFICIENCIES OF SPECIAL DRAFT TUBES

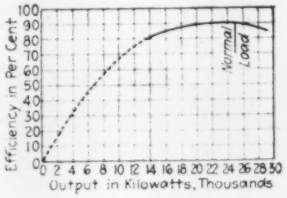


FIG. 4. EFFICIENCY DIAGRAM FOR STATION NO. 3

in synchronism with it. At the same time the Gibson apparatus makes its record of the changes of pressure that occur in the penstock. From such a record may be calculated the mean velocity of the water in the penstock prior to the shutdown. Having determined the velocity, the discharge is readily computed, and after allowing for the known efficiency of the generator, the turbine efficiency is calculated in the usual manner.

From these tests it has been determined that the turbine efficiencies are in excess of 80 per cent from half gate to full load, reaching in each case a maximum of 93 per cent. The average combined efficiency curve including losses in penstock, Johnson valve, turbine and generator, is shown in Fig. 4. From this curve it will be noted that at normal load the efficiency from forebay to switchboard is 90 per cent.

Details of the I. P. Morris Unit

By Lewis F. Moody

Assistant to Vice President and Consulting Engineer, William Cramp & Sons, S. & E. B. Co., Philadelphia, Pa.

TWO of the three turbine units in this installation were furnished by the I. P. Morris Department of The William Cramp & Sons Ship & Engine Building Co. These units involve in their design the most recent developments in this field of engineering, and include a number of interesting departures from previous practice. Every effort has been made by the builders of the turbines to provide the greatest possible reliability in operation and maximum obtainable efficiency.

The turbines are of the vertical shaft single-runner type, having cast-iron volute casings imbedded in the concrete of the power-house substructure. They operate at 150 r.p.m., and are designed to develop maximum efficiency when operated under an effective head of 213.5 ft. and discharging 1,500 sec.-ft. of water.

Taking up first the hydraulic features, it may be noted that the engineers of the power company working in co-operation with the engineers of the turbine builder have exercised every care to provide an efficient water passage from the point of entrance from the forebay into the penstock to the final point of discharge of the draft tubes into the tailrace. Preliminary tests have shown a very small loss of head in the penstocks, and indicate that excellent hydraulic conditions have been secured at all points in the water passage.

Before entering the turbine the water passes through a large Johnson valve. In passing through this valve the velocity of the water is gradually increased, the penstock diameter being reduced at the outlet of the valve to a value equal to the diameter of the inlet to the turbine casing so that the velocity of discharge from the Johnson valve is maintained at a constant value between the valve and the turbine. Owing to the construction of the turbine casing of cast iron, it has been possible to preserve smooth surface contours for the internal surfaces of the casing and to avoid all local disturbances in the flow due to abrupt changes of direction or irregular surfaces.

The diameter of the intake to the turbine casing is 10 ft. 6 in., involving a velocity of 17.3 ft. per sec. and a velocity head of 4.66 ft., which represents 2.18 per cent of the effective head of the turbine. The casing is designed for a gradual acceleration of velocity as the water passes around the volute. The casing is stayed across the distributor by ten curved vanes cast integrally in a cast-steel speed ring forming a part separate from the casing and built in halves. The vanes of the speed ring line up with alternate guide vanes, when the guide vanes are in the normal gate position corresponding to maximum efficiency. There are twenty movable guide vanes or wicket gates.

Spreading Draft Tube

After passing through the runner, the water enters a straight conical draft tube of cast iron built in sections. From the lower end of the cast-iron section of draft tube the remainder of the tube is molded in the concrete substructure.

The draft tube is of a new type, the "spreading draft tube." The design of this tube has been carefully calculated and verified by many tests of experimental models both under conditions of smooth streamline flow in the water entering the tube and under the actual conditions of flow experienced when the discharge is received from a runner. In the design of this tube it has been the object to turn the water as smoothly as possible along paths of gradual curvature from the vertical to the horizontal direction, and to preserve symmetry about the turbine axis to a point where the velocity has been reduced to so low a value that only a small amount of velocity head remains in the water. Care has been taken to maintain a gradual decrease in the velocity by properly varying the transverse area of the passage at all points in the tube, in order under all conditions of operation to avoid the formation of eddies or disturbances at any point in the stream.

In many earlier installations under high and medium heads, particularly those involving long penstocks, the use of elbows in the draft tubes has resulted in severe vibration of the water column in the penstock and turbine. This vibration is believed to be due to the formation and breaking down of eddies in the draft tube. Another advantage of the form of tube used is the ability of such a draft tube to regain the energy of whirling components of velocity in the water leaving the runner. This property of the tube is of value in increasing the efficiency under partgate and overgate conditions, and in increasing the margin of power beyond the point of maximum efficiency of the turbine. Preliminary tests of the turbine indicate that both of

these advantages have been realized, and in actual operation the unit is remarkably free from vibration of any kind at all gate openings.

Cast-Iron Runners and Casing

The turbine runners are of cast iron, each one being in one piece. The runners are 10 ft. 6 in. in diameter in inflow. The specific speed of the runners is 158 metric or 35.5 in the foot-pound system. The throat of the runner or the diameter inside of the band is 9 ft. 10½ in. At normal discharge the corrected velocity of the water leaving the runner and entering the draft tube is 21.3 ft. per sec., corresponding to a velocity head of 7.0 ft., or 3.3 per cent of the effective head on the turbine.

Although the most valuable feature of the new draft tube is probably the elimination of vibration just mentioned, it is also estimated that this tube will improve the efficiency over that which could be obtained with tubes containing elbows by from 0.3 per cent to 0.4 per cent in the efficiency of the entire turbine under conditions of normal gate operation. The same type of draft tube when applied to turbines of high specific speed would, of course, be capable of producing much greater increases in efficiency than is possible in these turbines, which are of moderate specific speed. The final discharge velocity from the draft tube at normal gate is approximately 5 ft. per sec., corresponding to a velocity head of 0.175 per cent of the effective head on the turbine.

The turbine casing is of cast iron built in six sections. The use of a cast-iron casing, although involving a somewhat increased cost as compared to the use of plate steel, furnishes a superior design in the following repects: The casing is absolutely rigid and can be imbedded in the concrete so as to form a part of the substructure, and can be relied upon to transmit the loads imposed upon it without distortion. The resistance of cast iron to corrosion insures a long life for the casing, which is a matter of importance since the casing could never be replaced without dismantling all of the surrounding concrete substructure. The casing can be depended upon to be tight throughout the life of the plant, and there is not likelihood of local points of weakness being developed as in the case of a riveted structure. The casings have been subjected to hydrostatic pressure tests in the shops to a pressure of 120 lb. per square inch., which is well in excess of any presure to which the casings can be subjected in operation during a quick grate closure resulting in water hammer. Another advantage of a cast-iron casing is the ease and speed with which it can be erected in the field. The heaviest section of the casing weighs slightly less than 60,000 lb.

The movable guide vanes are of cast steel, cast integrally with their stems. The guide-vane stems turn in bronze bushed bearings in the distributor plates and head cover.

Operating and Lubricating Mechanism

The operating mechanism is unusually rugged throughout, the guide-vane stems being of large diameter, the cast-steel operating ring extremely rigid; and the levers and other parts of the mechanism being made sufficiently strong to render the failure of any portion of the mechanism an unusual occurrence even when trash becomes lodged between two guide vanes and the entire governor power is concentrated on two vanes. Renewable breaking links are provided to protect the other parts of the mechanism in case undue load should occur. The operating ring is supported on a ball bearing. It is turned by two operating cylinders which are bolted to pads or brackets cast on the turbine casing, thus making the entire operating mechnaism self-contained with the turbine. The connecting rods between the operating ring and pistons are provided with adjustable ends of similar design to the connecting-rod ends used in steam-engine practice so that any wear may be taken up and lost motion avoided in the operating mechanism. The piston rod is provided with a bronze sleeve where it passes through the stuffing box in the cylinder head, and the rod is guided by a bearing supported in a guide bracket bolted to the cylinder.

For the lubrication of the operating gear, the Taylor system of lubrication is used, consisting of a central

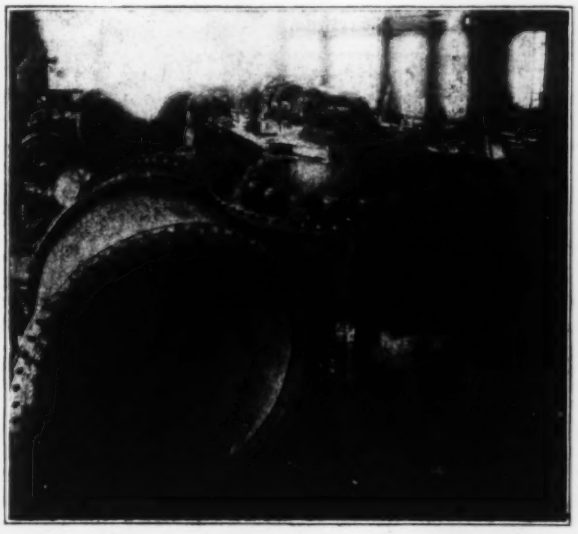


FIG. 5. I. P. MORRIS TURBINE ERECTED IN SHOP

grease gun supplying a system of piping leading to each individual bearing, so that the admission of grease or oil can be controlled to each bearing individually by turning a separate cock at each bearing, the lubricant being forced into the bearing by air pressure admitted to the grease gun.

A special design of runner seals have been used to reduce the leakage around the runner and to assist in making the thrust relief effective. The seals are of the multiple or labyrinth design in which a series of contractions are interposed in the path of the leakage water alternating with enlargements in which the velocity of the leakage flow produced eddies and reduces the quantity escaping. This provision although affecting only a small percentage of the total energy when the turbine is new will be of material advantage in preventing a serious deterioration in efficiency after the unit has been some time in operation. The leakage above the runner passes through a cored space in the turbine head cover and is vented through the runner hub, discharging into the center of the draft tube. The seals are formed in renewable wearing plates.

The distributor plates above and below the movable guide vanes are also renewable. At discharge from the runner the draft tube is lined with a renewable ring which may be replaced if corrosion should occur during the life of the unit.

The main shaft of the turbine is supported by a lignum-vitæ guide bearing in the turbine-head cover, and the shaft is provided with a renewable bronze sleeve where it passes through the bearing, seal and stuffing box. The bearing shell is split both horizontally and vertically to facilitate its removal. The equipment furnished with the turbine includes a set of brakes arranged to act upon the lower face of the generator rotor. These brakes consist of a series of shoes actuated by plungers, operated by air pressure. The operation of the brake is controlled from a cock mounted on the hand-control stand located close to the governor.

Governor Regulation

The turbine is regulated by a governor of the I. P. Morris double floating-lever type, belt driven from the main shaft of the turbine. The governor proper or the actuator and the separate hand-control stand are located on the gallery at the elevation of the thrust bearings mounted on top of the generators. The unit can therefore be operated from this point.

In order to avoid running a number of lines of the large governer piping from the operating cylinders up to the gallery with a consequent increase in the length of fluid column between the governor and the cylinders, resulting in increased inertia of this fluid column, the main governor valves are separate from the governor and placed on the level of the generator-room floor so that they are immediately above the operating cylinders.

The Taylor control system is used with these units. This involves the use of fluid-operated plunger valves of the Johnson type, by the operation of which the operating cylinders can be connected to either the governor or to the hand-control system. The plunger valves are located in a common casing with the main governor valve, which is located directly below the actuator, and the same casing supports a lower guide bearing for the governor spindle. All of the valves for shifting from governor to hand control and back again are controlled by a single lever mounted on the governor stand on the gallery. By a single throw of this lever a special control cock is operated which admits pressure to the plungers of the various Johnson valves and properly shifts the pressure and return connections from governor to hand control or vice versa. By this system, failure of the operators to manipulate the valves in proper sequence is avoided, thus avoiding loss of control of the unit between the time of its being taken off the governor and put on hand control. The entire operation can be carried out very quickly, which is frequently a matter of importance in station operation, and all loss of time required to open and close the series of valves by hand is avoided.

The separate hand control provided in addition to the governor contains separate valves and restoring mechanism by which the turbine gates can be operated through the pressure system, the gates being automatically maintained in any position corresponding to the setting of the hand wheel. By this means the unit can be operated on hand control with the governor and all of its valves thrown out of operation and made available for inspection and repair. A clutch is provided by which the governor head may be put out of operation without shutting down the unit so that all parts of the centrifugal mechanism can be made accessible. The centrifugal governor head is extremely powerful and is not influenced by slight changes of friction of the governor parts or other variation of conditions.

Among the members of the Cramp company's organization who have been responsible for the design of this turbine installation may be mentioned: H. Birchard Taylor, vice-president; John Overn, Jr., manager, I. P. Morris Department; Frank H. Rogers, hydraulic engineer; and R. E. Brunswick Sharp, assistant hydraulic engineer.

Design of the Allis-Chalmers Unit

By W. M. White

Manager and Chief Engineer, Hydraulic Dept., Allis-Chalmers Mfg. Co., Milwaukee, Wis.

THE hydraulic turbine forming Unit No. 16 of the extension to Station No. 3 is rated at 37,500 hp. when operating at the normal speed of 150 r.p.m. under the normal operating head of 214 ft. and is designed to use 1,500 sec.-ft. when operating at best efficiency. The unit has recently been put under full load and actually develops at normal head and speed a capacity of 40,000 hp.

Unit No. 16, which is the first one installed and the first placed in operation, was designed and built by the Allis-Chalmers Manufacturing Co. at Milwaukee, Wis. Since the design of the entire Allis-Chalmers hydro-electric unit was under the control of a single group of engineers it was possible to consider the structure properly as a homogeneous unit and correctly proportion and correlate the various elements. Such procedure was followed so that the completed machine possesses a unity not frequently met with in hydro-electric practice.

The design of the turbine embodies several new features of interest to water-power engineers, particularly the form of casing, governor, interconnecting barrel, brake support and draft tube used.

The water for the units in the new station is taken from the old Hydraulic Power Co. canal and is delivered to the unit through a 15½-ft. diameter penstock and controlled by Johnson valve located near the turbine. The discharge diameter of the Johnson valve is 10 ft. and to this is connected a short section of the feeder pipe which increases in diameter to 11 ft. at the entrance to the plate-steel circular section spiral casing surrounding the turbine and supplying water through guide vanes to the runner. The casing is made up of a multiplicity of conical sections of plate steel of gradually reducing cross-sectional area around the turbine. The plates are riveted to the flanges of a cast-steel speed ring which forms an integral part of the casing and the foundation of the turbine parts. The thickness of the plate steel of the casing at the 11 ft. diameter is ⅞ in. This thickness is reduced gradually to ½ in. at the smallest section of the casing. The roundabout seams of the casing are lap joint, double riveted and the longitudinal seams are lap joint, triple riveted, while the connection at the ends of the

plate to the cast-steel speed ring is made by a triple row of rivets. The rivets are countersunk on the inside

Plate-Steel Casing

Fig. 6 gives a view of the casing as erected in power house. There are about six thousand rivets in the casing which were driven by a No. 90 air hammer. The heads were formed by 1¼-in. by 1¼-in. snaps. The casing was calked inside and outside. It was concreted in without an hydrostatic test. No leaks of any kind have developed about the casing. Those who are familiar with plate-steel work will readily understand that plate thickness of 1¼ in. can be readily worked so that a casing substantially of the size shown could be designed and constructed for heads up to 350 ft. without unduly stressing the material or sacrificing tightness and without going beyond normal practice obtaining in plate work of this nature. Above all, however, all uncertainties as to casting strains are eliminated and only strictly dependable material used. On account of the simplicity of the construction and low cost relative to cast iron this casing was made larger in diameter so as to reduce losses due to friction, thereby tending toward higher turbine efficiency.

Water is controlled from the casing to the runner by means of twenty cast-steel guide vanes. Each vane has pivots on each end for the support of the vane and for controlling its position. Each vane is operated by means of a lever secured to the extended pivot through a link connecting to the shifting ring located on the outside of the turbine. Each guide vane is held in position by two thrust bearings, one supporting the weight and the other resisting the upward hydraulic thrust of the stem through the stuffing box when the unit is in operation.

The thrust bearings are so adjusted that the guide vane is held in a central position between the distributer plates with fixed and equal clearances on each end of the vane; consequently there is no friction and no wear between the ends of the vanes and the distributer plates.

A stuffing box is provided about each vane stem and water is thereby prevented from flowing through the bronze bearings supporting the vane—a specially desirable feature in that silt and sand which would tend to rapidly wear the main supporting journal is not carried between the surfaces by leakage. Levers are keyed to the vane stems and attached to the shifting ring through an adjustable connection and links having a cross-sectional area so proportioned as to fail before a guide vane or its stem is strained beyond its elastic limit should a guide-vane passage become obstructed. Heavy plate-steel wearing plates line the distributer plate at each end of the guide vanes. Wearing bushings are embodied on the upper and lower distributer plates opposite corresponding wearing rings placed on the upper crown plate and the lower band of the runner.

Runner of One Casting

The runner is of grey iron in one casting made by means of fitted cores and is bolted to a cast-iron hub which is keyed to the tapered end of the turbine shaft and held in place by ring, key and keeper. Either a taper connection as here used or a forged flange on the end of the shaft upon which the runner is mounted makes a good construction, but where a water bearing is used, with a bushing on the shaft, the construction here used is the better one, as it enables the bushing to be made solid and therefore of less thickness and results in the lower peripheral speed for a given unit. The diameter of the shaft at the turbine guide bearing is 28 in.

The turbine guide bearing consists of lignum vitæ fitted into cast-iron blocks having the end grain of the lignum vitae toward the shaft. The bearing of this unit is 5 ft. in length and is made in two parts for convenience in dismantling and ease of handling. The bearing is lubricated by water supplied from the headrace through fine strainers. Connection is also made

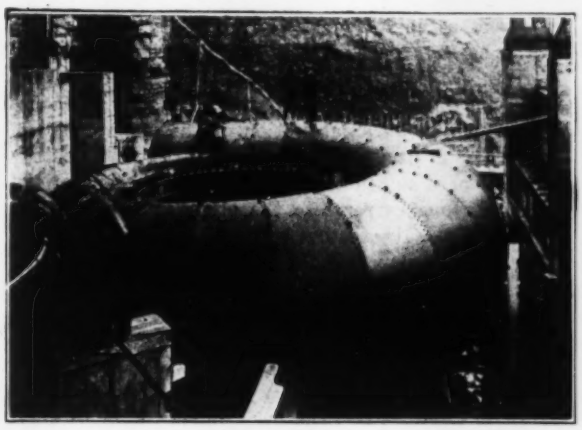


FIG. 6. ALLIS-CHALMERS TURBINE CASING ERECTED IN POWER HOUSE

to the city mains for use of filtered water during times when headrace water contains considerable silt and sand.

A feature which distinguishes Unit No. 16 from the other units is the cast-iron pit liner forming the turbine pit and transmitting the weight of the generator through this pit liner directly to the speed ring and through the vanes of the speed ring to the foundation. From a mechanical and design standpoint the value of this barrel can hardly be overestimated, tying together as it does all the major elements in which forces originate and from which they have to be transmitted. Interalignment of main working elements, i.e., generator, turbine and regulating cylinders, is permanently insured regardless of the behavior of the surrounding concrete.

The guide vanes of the turbine are controlled by means of water pressure on two pistons connecting to opposite sides of the shifting ring, inclosed by cylinders, mounted directly on the outside of the supporting barrel or pit liner. The mounting of the operating cylinders on the outside of the pit liner is unique and affords a clean design of the pit, as it keeps the governor pipes outside of the turbine chamber. This feature of design enables the cover-plate of the turbine to be removed by merely removing the pistons and crossheads and without disturbing the cylinders and governor piping.

An interesting feature of design is the governor equipment especially designed for this unit. The guide vanes are shaped to cause the smoothest flow from the casing to the runner and the position of the stems on the guide vane body is such that the vanes will be

in hydraulic balance when about one-third open. They will require considerable force to close the vanes from that point and considerable force to hold them in wide-open position under the high velocity of the water between the vanes, but both forces are minimized. The operating cylinders are 32 in. in diameter and the governor is designed for a normal operating pressure of 125 lb. so that the two pistons are capable of exerting 150,000 ft.-lb. when moved the full stroke from one end of the cylinder to the other. With a difference between the sides of the pistons equal to an operating pressure of 200 lb. and consequently by the usual method of rating governors, the governor of this unit has a capacity of 240,000 ft.-lb.

The operating pressure to the cylinders is controlled by means of a four-ported 12-in. diameter piston valve located in the governor stand. The pipe connecting the ports from the piston valve to the operating cylinders is 6 in. in diameter. This large diameter valve is necessary so that an axial motion of the valve of $\frac{1}{2}$ in. will at normal pressure afford the area of opening required to pass sufficient water to force the operating pistons from one extreme to the other in two seconds of time. The main piston valve, above described, is moved axially by varying the pressure on each end of its body by means of another four-ported 3½-in. diameter piston valve and this piston valve in turn is moved axially by varying the pressure on each end of it by a $\frac{3}{4}$-in. diameter pilot valve. By means of the smaller piston valve the size of the pilot valve and consequently the force to be exerted by the flyball is materially reduced over what it would have to be were the valve omitted.

The edges of the ports and the edges of the piston valves and the pilot valves are so carefully made that under normal operating pressure that is practically no lost motion between the axial motion of the pilot valve and the axial motion of the main piston valve. Under slow motion of the pilot valve the main piston valve will follow with a lag of less than 0.01 in. and under the axial movement of the pilot valve of $\frac{1}{8}$ in. in 0.01 in a second the maximum lag of the main piston valve would not exceed 0.02 in. By thus reducing the size of the pilot valve the force required to move it has been reduced to such an amount that it does not exert sufficient force upon the flyball mechanism to prevent the flyballs taking up their exact position corresponding to the speed of the unit.

It was thought that a turbine of this magnitude should be equipped with a flyball mechanism steady, positive and as near frictionless as possible. The directly connected flyballs embodied in the design of this turbine have met these conditions. The flyball mechanism consists essentially of a fixed collar clamped around the main turbine shaft and a movable collar held in position and away from the shaft by four sets of ball-baring toggle joints located 90 deg. from each other. Two sets of toggle joints as incorporated in the design act as flyball weights. The outward motion of these sets is resisted by two adjustable springs being connected through ball bearings to each of the sets. Every toggle is fitted with a type of ball bearing preventing axial motion and consequently each set of toggles prevents endwise motion of the floating ring with respect to that set of toggles and as the two sets of toggles are at 90 deg. the movable collar is rigidly supported and prevented from moving in any direction except axially with respect to the main turbine shaft. It is a unique feature in governor flyball design that the floating collar has no support except through the motion giving mechanism. This design of flyball, therefore, affords a means of imparting to the pilot valve such motion as is caused in the flyball by the variation in speed of the main turbine shaft and thus is avoided the disturbing motion introduced by belts and gears when the usual type of flyball is used. The motion of the movable collar located around the main shaft is transmitted to the pilot valve by means of levers, reach rods through sliding shoes resting upon an oil-inclosed bearing surface at the upper side of the movable collar.

The governor has been adjusted to close the turbine gates from wide open to completely closed in three seconds of time and to open them from closed to wide-open position in four seconds of time. It is expected that the rise of speed with 40,000 hp. suddenly thrown off the unit will not exceed 25 per cent. The regulation of the unit and the normal load conditions are remarkably steady and positive.

Hydraucone Draft Tube

The engineers of the Allis-Chalmers Manufacturing Co. at the time of first putting this unit into operation made a remarkable demonstration to show their confidence in the apparatus. The initial start of the unit was made solely from the station switchboard by closing the switch operating the control motor on the governor which moved the pilot valve causing the main turbine gates to open. This admitted water from the casing to the runner setting the unit in motion. When the unit had attained a speed of 90 r.p.m. the weights of the flyballs came into play and manipulated the pilot valve and regulated the unit perfectly at that speed. Without any adjustment in the governor being made the governor motor was manipulated from the switchboard and the unit brought up to a speed of 150 r.p.m. within ten minutes of the initial starting. The governor regulated the unit positively, steadily and perfectly under that speed.

One of the greatest losses in hydraulic turbines is in the energy discharged from the runner. Efforts have heretofore been made to utilize this energy by transforming the velocity into pressure for maintaining below the runner greater vacuum than would normally result due to the elevation of the runner above tail water level, and thus in effect increasing the effective head on the turbine. The size of units in water-power development have been increased until the diameter of the runner is large compared to the distance from the runner to the tail water level so that in large units it has not been found feasible to design the draft tube on account of the short radius of bend which will most efficiently utilize the energy discharge from the runner with an excavation of tailrace such as the owner is usually willing to make.

In Fig. 2 is an illustration of one form of "hydraucone regainer."

The word "hydraucone" has been coined to express the shape which a jet of water takes upon striking any given surface and includes the form from the point where this stream begins to make its turn to the end of the curvature. Consequently a hydraucone regainer

was used which is a chamber having a general form which the jet of water would take upon striking the impinging surface except that the capacity of the chamber is gradually greater in area in the direction of flow than that required to just close the hydraucone. The radially extending passage also affords a means of regaining for useful effect the whirl of the water as it leaves the runner at partial loads. It is not within the scope of this article to discuss at length the hydraucone.

This unit has not been in succesful operation for a sufficient length of time to show that all of the new elements embodied in its design are working as planned and that the complete unit is an unqualified success.

Crushing Strength of Southern Pine at Angles to Grain

By Q. C. Ayres

Assistant Professor in Civil Engineering, University of Mississippi, Oxford

PRACTICALLY all designs in wood involve problems vitally dependent on the compressive strength of timber on surfaces oblique to the fibers. There is marked disagreement, however, as to the behavior of the wood under such compression. A series of tests recently conducted at the University of Mississippi on specimens of Southern pine gives data tending to clear up the existing uncertainties.

Southern yellow pine, or Southern pine, as it is called under the modern standard classification, includes the pines hitherto known as long leaf (*Pinus palustris*), short leaf (*Pinus echinata*), loblolly pine (*Pinus taeda*),

FIG. 1. CUTTING SEVEN TEST SPECIMENS FROM A STICK

Cuban pine (*Pinus heterophylla*), and pond pine (*Pinus serotina*). The test specimens, cut from pieces selected from a dealer's stock, probably included wood from several of these species, but the consistency of the results obtained in the tests indicates that all Southern pines behave substantially alike in the matter of variation of crushing strength with direction of loading.

Stock pieces were used for the tests in order to reproduce as nearly as possible conditions met in practice. From each stick, dressed to about 3½ x 1¼ in., seven blocks 2½ x 2½ x 1¼ in. were cut as indicated in Fig. 1, each of the seven having its test axis at a different angle with the grain, these angles being 0°, 15°, 30°, 45°, 60°, 75° and 90°. The specimens were dressed on all four sides. The sticks from which

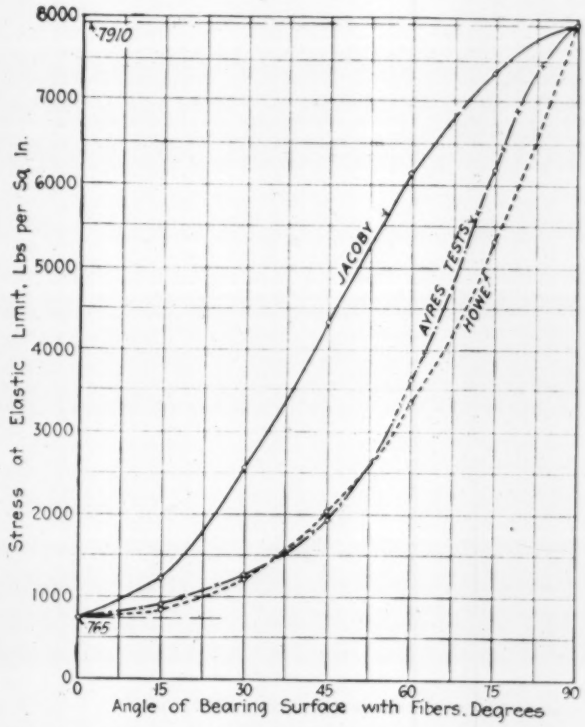


FIG. 2. RESULTS OF TESTS ON SOUTHERN PINE, AND PLOT OF JACOBY AND HOWE FORMULAS FOR SAME TERMINAL VALUES

they were cut rated 70 per cent heart and 30 per cent sapwood, were free from all visible defects, and were thoroughly air-seasoned. It was found that their average moisture content was 8.15 per cent, the weight of the wood 35 to 42 lb. per cubic foot, the density 7 to 14 rings per inch, and the percentage of springwood 70 to 90 per cent.

These test blocks were crushed in a 50,000-lb. Olsen hydraulic testing machine. Load was transmitted directly to the wood by the spherical bearing plate of the machine, and no deflectometer was used, as it was desired to avoid influencing the results by applying load over only part of the bearing face. The load at the elastic limit of the specimen was determined by observing the motion of the arrow on the dial of the testing machine. In each case the arrow responded to a uniform applicatior of the load by a uniform advance on the dial until the wood began to yield, when it advanced fitfully (with more rapid application of the load) until final failure of the specimen. The value registered by the arrow at the point where its motion ceased to continue uniform was taken as the elastic limit load. These values are assembled in the accompanying table.

CRUSHING STRENGTH OF SOUTHERN PINE OBLIQUE TO THE GRAIN—AYRES

Angle of Surface with Fibers	Load at Elastic Limit—Pounds per Square Inch											Limiting Load by Formula	
	Set 1	Set 2	Set 3	Set 4	Set 5	Set 6	Set 7	Set 8†	Set 9†	Set 10†	Mean	Jacoby	Howe
0°	694	703	742	731	721	654	700	1,023	806	873	765	(765)*a*	(765)*a*
15°	758	811	789	809	895	838	800	1,122	1,220	1,126	915	1,245	845
30°	937	1,102	1,011	987	1,219	1,176	1,067	1,557	1,714	1,818	1,260	2,550	1,235
45°	1,468	1,553	1,592	1,568	1,845	1,845	1,500	2,367	2,691	2,727	1,915	4,340	2,030
60°	3,717	3,425	3,678	3,048	3,905	3,412	2,881	3,491	4,275	4,296	3,615	6,125	3,360
75°	6,904	6,175	6,871	6,259	6,934	6,150	*	4,897	6,130	5,048	6,150	7,430	5,295
90°	7,908	7,718	7,922	7,710	8,448	8,664	*	7,636	6,580	8,582	7,910	(7,910)*a*	(7,910)*a*

* Results discarded; unreliable. † Sapwood. *a* Taken from test results and used as constants in the formula.

For the specimens tested parallel to the grain, the failure was by sliding along a plane oblique to the grain; *for all other specimens failure occurred by shear along a plane parallel to the fibers.*

Two formulas for the strength of wood oblique to the grain have been proposed, the first developed theoretically by Prof. H. S. Jacoby, the second derived empirically by Prof. M. A. Howe:

Jacoby, $n = p \sin^2\theta + q \cos^2\theta$

Howe, $n = q + (p - q)\left(\frac{\theta^\circ}{90^\circ}\right)^{5/2}$

where p = allowable intensity of stress in end bearing.

q = allowable intensity of stress in cross bearing.

n = allowable intensity of stress on a surface inclined at angle θ with the grain.

Values of n computed by these two formulas are appended to Table I, using average values of p and q as found from the tests (elastic-limit values). There is a fair agreement between the test values and those computed from the Howe formula, although the latter is based on load at a fixed deformation and not on elastic-limit load.

A better comparison of the test results and the formulas is presented by the curves, Fig. 2, representing the average values given in the table. It will be noted that Howe's formula gives values which err on the side of safety; in view of this fact, it may be considered the best and simplest expression that can be devised for Southern pine, and accordingly is recommended for use in dealing with that material.

At present working stresses of 350 lb. per square inch in cross bearing and 1,600 lb. per square inch in end bearing are commonly used. The present results would indicate that, pending further investigation, a decrease in the cross-bearing value and an increase in the end-bearing value are advisable, the amount of change depending on the factor of safety chosen.

Inclined-Bearing Tests on Douglas Fir and White Pine

BY THOMAS R. SIMPSON
Berkeley, Cal.

BECAUSE of the marked difference between the Jacoby and Howe formulas for permissible values of bearing pressure on wood when the surface is inclined to the grain, and because of divergences between the available test data, a new series of tests

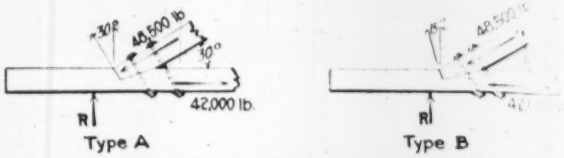


FIG. 1. INCLINED BEARING AT TRUSS JOINTS

was made by the writer during the past year at the University of California. Douglas fir and California white pine were the materials. A rather remarkable result developed, namely, a close check with the Jacoby formula when the elastic-limit load was taken as criterion of permissible value, and approximate agreement with the Howe formula when a fixed depth of indentation was taken as criterion. The two woods behaved in substantially identical manner so far as the law of variation of load with angle of grain is concerned.

To indicate the importance of the subject, it may be remarked that the joint details of wooden trusses depend to a very great extent on the values adopted for inclined bearing pressure, as compared with those for end-grain and cross-grain compression. The formula of Prof. M. A. Howe, which incidentally is recommended in the latest edition of Kidder's Handbook, gives values considerably below those of Prof. H. S. Jacoby's formula, and in consequence some truss details are virtually impossible under the Howe formula, while they have satisfactory proportions under the Jacoby formula. Thus, referring to an end joint of the form sketched

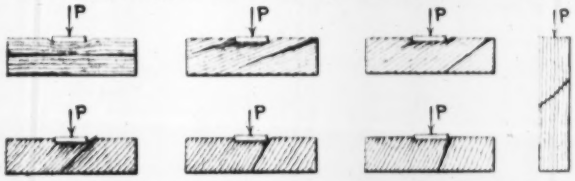


FIG. 2. MANNER OF FAILURE IN TESTS

in Fig. 1, with stresses as shown, each member made of an 8 x 8-in. stick, the depths of cut required under the Jacoby formula would be 4.87 in. in Type A and 3.86 in. in Type B; under the Howe formula, the depths of cut would be 8 in. and 5.23 in. respectively so that the proposed form of joint would simply not be feasible. There are other cases in which the formula used plays an almost equally important part, and in any event the design of washers, lugs, and other metal parts bearing on inclined surfaces is materially affected.

Referring to the origin of the two formulas, it should be said that the Jacoby formula was derived from theoretical considerations while Professor Howe derived his formula from tests, taking the values of loads giving a deformation of 0.03 in. However, in pure end-grain compression a load giving a deformation of 0.03 in. is considerably beyond the elastic limit for most woods, while in cross-bearing the load for 0.03-in. indentation is usually below the elastic limit.

To furnish a basis for choice between the two for-

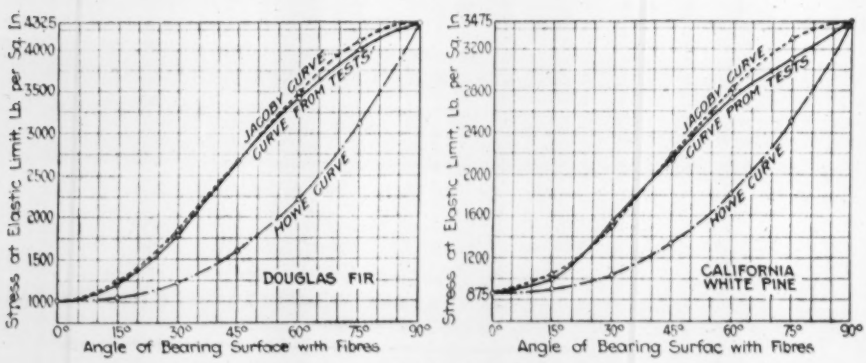


FIG. 3. ELASTIC LIMIT VALUES CHECK JACOBY'S FORMULA

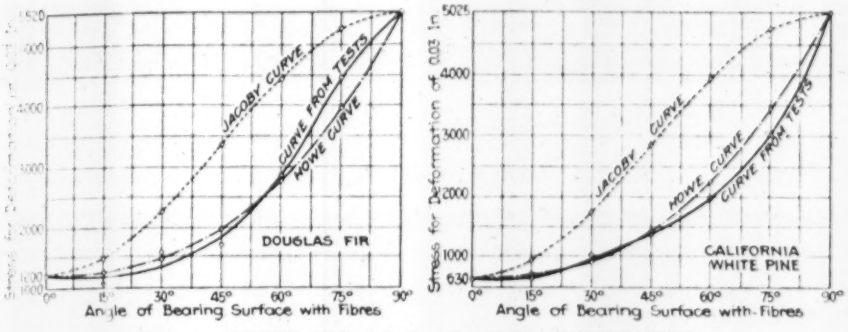


FIG. 4. CURVES FOR LOAD AT 0.03-IN. INDENTATION

mulas, specimens of douglas fir and of California white pine, 2 x 2 x 8 in., were used, all carefully selected for uniformity, and oven-dried for 24 hours to insure uniform moisture content. All wood was straight-grained and free from visible defects. Load was applied on the flat face of the specimen, but only over the middle 2 in. of length, the loading being done by a strip of ½-in. tool steel 2 x 12 in., laid across the specimen, directly under the pulling head of the machine. Thus an area of 4 sq.in. (out of 16 sq.in.) was loaded. Deformations were measured by a compressometer for the specimens in end bearing, and by a multiple-lever deflectometer in the other cases.

Stress-strain curves were plotted from the results of the tests, and the loads at elastic limit were obtained from these curves. Very satisfactory straight lines up to the elastic limit or yield point were obtained in practically all of the tests. Readings being taken at intervals of about 500 lb. load increment, the load for a deformation of 0.03 in. could also be read off accurately. Both resulting values are given in the tables herewith. Five specimens were tested for each angle of bearing, and the mean values are given; the variations recorded within any set of five were relatively small.

INCLINED-BEARING TESTS OF DOUGLAS FIR AND CALIFORNIA WHITE PINE

DOUGLAS FIR

Angle of Surface with Fibers	Mean Load at Elastic Limit, Lb. per Sq. In.	Allowable by Jacoby Formula	Allowable Load by Howe Formula	Mean Load for 0.03-In. Indentation, Lb. per Sq. In.	Allowable by Jacoby Formula	Allowable Load by Howe Formula
0°	1,010	1,010	1,010	1,200	1,200	1,200
15°	1,205	1,230	1,050	1,058	1,480	1,249
30°	1,785	1,835	1,220	1,572	2,280	1,475
45°	2,670	2,665	1,595	1,695	3,360	1,965
60°	3,440	3,495	2,210	2,840	4,440	2,765
75°	4,015	4,115	3,120	4,465	5,250	3,950
90°	4,325	4,325	4,325	5,520	5,520	5,520

CALIFORNIA WHITE PINE

Angle of Surface with Fibers	Mean Load at Elastic Limit, Lb. per Sq. In.	Allowable by Jacoby Formula	Allowable Load by Howe Formula	Mean Load for 0.03-In. Indentation, Lb. per Sq. In.	Allowable by Jacoby Formula	Allowable Load by Howe Formula
0°	875	875	875	630	630	630
15°	1,000	1,050	905	653	926	680
30°	1,540	1,500	1,040	1,010	1,728	910
45°	2,160	2,175	1,335	1,378	2,828	1,305
60°	2,750	2,825	1,815	1,965	3,926	2,220
75°	3,110	3,310	2,520	3,005	4,755	3,420
90°	3,475	3,475	3,475	5,025	5,025	5,025

The initial failure of all specimens was in crushing of the fibers, while final failure took the form of shear along the fibers except in the end-bearing specimens.

In the accompanying Figs. 3 and 4 the results of the tests are plotted, both elastic-limit values and values at 0.03-in. indentation. Together with these test curves are plotted the Jacoby and the Howe formulas, using terminal values corresponding to those given by the tests.

Conclusions—The loads for 0.03-in. indentation correspond closely with the Howe curve. Since such load is usually well above the elastic limit in end bearing, and below the elastic limit in cross-bearing, it would seem good practice to base the allowable bearing stress in engineering design upon the loads at the elastic limit. These latter correspond remarkably with Jacoby's theoretical formula.

As a result it is believed that while the mathematical analysis offered by Jacoby for the proof of his formula is not altogether satisfactory, yet the formula appears to express the proper relation between load and angle of grain, and should be used.

Use of Clinker Asphalt in England Lowers Paving Costs

THOUGH asphaltic concrete with clinker as coarse aggregate was used to some extent in England prior to the war, only recently, says *The Surveyor*, has the manufacture of clinker asphalt approached commercial perfection. The Borough of Hornsey is recognized as the leader in clinker-asphalt production, the special interest in the Hornsey material being the fact that it is composed entirely of clinkers derived from the borough dust destructor, which, after preparation to meet varied requirements of grading, is mixed with pure bitumen. Large areas of this clinker asphalt were laid in the Borough of Hornsey in 1914, but owing to war conditions the plant used was of a somewhat crude character, comprising open-hearth driers and a mixer driven by a small gasoline engine. Even during the war this method of manufacture was continued, but since the war certain changes in plant and methods have resulted in a considerable decrease in the amount of labor needed, and a noticeable increase in output.

It is pointed out that a conspicuous feature of clinker-asphalt road construction is the comparative freedom from waves which are so pronounced in certain bituminous construction. Lack of waving in clinker asphalt is no doubt due largely to the interlocking and absorptive character of the particles formed in the aggregate used.

The Surveyor further says: "In the earlier years the work was carried on at about half the cost of ordinary bituminous construction, and notwithstanding the enhanced cost of labor and materials it still shows a saving of upwards of 10s. per super yard as compared with other bituminous construction of not superior traffic-resisting qualities."

Utility of Hardwoods for Paving Shown in Comparative Tests

Relative Insusceptibility to Expansion Through Water Absorption, and Greater Strength Recommend Oaks and Hickory

By E. E. Butterfield
In Charge of Tests, Office of the President, Borough of Queens, New York City

DIFFICULTY in obtaining a reasonably uniform quality of creosoted pine paving blocks in 1918 led to an investigation of the physical properties of some American hardwoods with reference to their possible utility for wood block pavements. The widespread failure of creosoted pine pavements during and following the rainy spell in the summer of 1919 led to further examination of the physical properties of creosoted pine paving blocks for purposes of comparison with the hardwoods, as well as for the purpose of obtaining as much information as possible which would throw light on the causes and character of failure of the creosoted pine block pavements.

The 1918 specifications of the Borough of Queens called for creosoted pine paving blocks to comply with the following requirements:

Annual rings, per inch, average	Not less than 6
Oil, lb. per cu.ft	Not less than 20
Water absorption, 24 hr. at 100° F. followed by 24 hr. immersion in water, gain in weight	Not more than 5%

Samples from four deliveries under contract gave the following results:

	1	2	3	4
Oil, lb. per cu.ft.	14.8	18.3	13.7	18.9
Water absorption	5.5	5.0	16.7	5.8

The contractor made a request for re-sample and retest and a representative of the manufacturer was permitted to inspect the material and to select samples at the various points of delivery. The results of the tests on these samples were as follows:

Rings per Inch	Water Absorption, per Cent	Rings per Inch	Oil Lb. per Cu.Ft.
5	8.2	4	21.2
6	9.0	5.4	19.3
6.1	7.0	5.4	18.1
10.4	11.0	6.6	18.4
15.3	4.6	6.6	18.7
20.2	6.9	9.0	15.5
		9.0	18.0
		10.0	17.7
		10.2	21.2
		10.5	18.7
		12.5	19.4
		16.0	17.0
		16.6	11.5
		18.0	17.3
		21.1	20.4

Other blocks from the same deliveries showed water absorption varying from 5.9 per cent to 34.7 per cent.

It was impossible to arrive at a logical basis for appraising the value of the blocks for paving purposes inasmuch as none of the customary tests had any direct bearing on the strength, wearing qualities or durability of the material. Plans were therefore formulated for a series of physical tests such as are made on structural timbers with necessary modifications for the size, shape and function of wood paving block.

While this investigation was in progress numerous and widespread failures of creosoted pine block pavements occurred in the Borough of Queens, in January, 1919, and during the prolonged rainy season of the summer of 1919 and again during the period December 1919-February 1920. According to seasons and climatic conditions there were two different types of failure.

The winter failures occurred during a sudden drop in temperature to below the freezing point of water, which was preceded by a period of moderately warm damp weather. It would seem that the winter failures are due to the sudden expansion of water into ice, either water infiltrated beneath the blocks or water absorbed by them.

The other type of failure occurred at summer temperatures following prolonged rains. Failures of this type occurred on Jackson Ave., Long Island City, at many other points in the Borough of Queens, and in other cities throughout the East during the rainy spell in the summer of 1919. The summer failures occur at high temperatures under conditions favoring a maximum saturation of the blocks with water. Under such conditions the forces of expansion of the wood, of the oil and absorbed water are all acting in the same direction.

In both winter and summer failures the end result is the same, although produced suddenly at the low temperatures and occurring gradually at elevated temperatures during or following rainy spells. There are local areas of bulging, arching of the affected pavements, accompanied or followed by fracture of the blocks. On closer examination many of the fractured blocks present the appearance of having been sheared along the grain.

Failures of existing creosoted pine block pavements, laid under rigid specifications and with utmost care, following closely on our experience with the lack of uniformity of the available materials in 1918 gave rise to reasonable doubt as to the effectiveness of prevailing specifications and methods of tests in insuring satisfactory results with wood block pavements. Consequently, an attempt was made to select tests which would furnish more direct evidence of the wearing qualities and durability of the wood.

Selection of Tests

Laboratory tests on paving materials should conform as closely as possible to the practical working conditions to which the pavement will be subjected in actual service. It is not to be expected that there will be a direct relation or a parallelism between the results of laboratory tests and the results of practical service. The most that can be attained is an approximation, yet this approximation should be made as close as possible. The tests should represent essential independent variables. Irrelevant tests should be avoided, as should also tests which may tend to confuse issues by expressing similar results in different terms. The tests should imitate the forces or conditions to which the pavement will be exposed in service. The principal forces to which a pavement is subjected are compression, abrasion, impact and shear from traffic, externally, and compression and shear from expansion, internally. Tests to determine shearing strength, impact, and water absorption were selected as the most significant, important and practicable. Compressive strength along the grain is of little practical value inasmuch as a pavement is rarely, if ever, subjected to a static load of the magnitude of the compressive strength of wood.

In expansion, a wood block pavement is subjected to compression on all sides in a series of horizontal planes.

When slight arching of a row or a stretch of blocks exists, the direction of the grain forms an acute angle with the perpendicular, and the horizontal force may be resolved into rectangular components, one acting in the direction of the grain and the other across the grain. According to the numerical values of the horizontal force and its components with and across the grain, the blocks may fail in shear along the grain or in compression across the grain. Blocks taken from areas of failure previously mentioned exhibited a clean shear along the grain, regardless of the season of failure, whether winter or summer. If the pavement becomes raised from the foundation either from bulging and arching or from existing irregularities or depressions in the foundation, then the impact of traffic would also tend to produce a mechanical shear in the unsupported areas.

The tests for shearing strength were made on specimens prepared in the same manner as for tests of single shear on structural timbers and as described by Samuel J. Record in "Mechanical Properties of Wood," 1914 Edition, p. 107.

Impact Tests

To test for impact in the laboratory the blows are delivered by a hammer which strikes the same spot repeatedly until failure occurs. In actual service the blows are delivered by the wheel of a rapidly moving vehicle so that, in addition to the simple impact delivered from varying heights according to smoothness of the pavement, there is the important factor of velocity. The laboratory impact test is a rather feeble substitute for the glancing blow delivered by a 14-ton truck traveling at from 15 to 30 mi. an hour. The impact tests were made on cylindrical cores 2.5 cm. in diameter by 2.5 cm. in height, of the same form and size as cores of granite block or broken stone selected for the toughness or impact test. The cores were tested in an impact machine with a 2-kilogram hammer raised 1 cm. each blow. The figures represent the number of blows as well as the height of the hammer at which failure occurred. The impact test performed in this way is not the same as the method of testing impact on structural timbers but it is adequate for comparative purposes and possesses the advantage of permitting a comparison between the impact or toughness values of wood and stone blocks.

Tests for water absorption are important not so much for the purpose of fixing an arbitrary value for the permissible absorption of water as for the purpose of taking into account the expansion caused by the absorption of water. In order to obtain some information on the magnitude of the expansion caused by water absorption three creosoted pine paving blacks, 8 in. x 3 in. x 3 in., were maintained in an atmosphere saturated with water vapor for one year, measured accurately in three dimensions at room temperature, dried in a vacuum at 140 deg. F. to constant weight, cooled to room temperature, and again measured. The blocks were at no time in contact with or immersed in water and the changes in volume represent the changes produced by the absorption of water vapor at constant temperature. Tests upon these three blocks gave the following results:

Moisture Content (Loss in Weight), per Cent	Change Length, In.	Change Width, In.	Change Depth, In.
12	0.234	0.077	0.000
14.8	0.496	0.200	0.004
14.9	0.367	0.112	0.005

A change in length of 0.234 and 0.496 in. in an 8 in. x 3 in. paving block is equivalent to 36 and 82 in. per 100 lin. ft., and a change of 0.077 and 0.2 in. in width is equivalent to 32 and 85 in. per 100 lin. ft. Changes of such magnitude are astonishing when one considers that they were due solely to differences in water content at constant temperature and that consequently the results are not in any way related to the expansion of the wood at different temperatures. The coefficient of expansion of wood across the grain, that is in the length and width of wood paving blocks, is 0.0000544 per degree Fahrenheit, according to Record, or 0.6528 inches per 100 deg. F. per 100 lin. ft. The coefficient of expansion (cubical) of creosoting oils is 0.00044 per degree Fahrenheit, or about 4 times the coefficient of expansion of water. While there is no known law by which the expansion of absorbed water, the expansion of the wood, and the expansion of the oil may be computed, it is readily conceivable that these three factors working in the same direction may give rise to forces far in excess of the crushing strength of the wood across the grain. In the future, figures for water absorption should be supplemented by the measurement of the expansion produced by the water absorption.

In the following series of tests the water absorption was determined by 24 hours' immersion, preceded by a period of drying at 100 deg. F. for 24 hours. The figures in the table represent percent gain in weight.

TABLE I.

Kind of Wood	Shearing Strength Lb. per Sq.In.	Impact (Toughness)	Water Absorption, per Cent	Linear Expansion Across the Grain, Accompanying Water Absorption — In. p. F.
White Oak	1,620	36	6.0	0.080
Red oak	1,158 radial	36	18.8	0.254
Chestnut Oak	1,364 tangential 1,598 radial	40	8.2	0.193
Post Oak	1,230 radial	41	8.6	0.162
Hickory	1,465 tangential 1,481 radial	46	8.4	0.219
Maple	1,163 radial	49	13.6	0.330
Yellow Birch	1,183	37	12.3	0.223
Red Birch	999	39	13.5	0.225
Southern Pine, untreated	1,258 tangential 1,666 radial	22	11.2	0.239
Southern Pine, creosoted	1,237 radial	13	10.2	0.151

All of the hardwoods tested appear stronger on impact than creosoted pine. Untreated pine is much more resistant to shear and impact than creosoted pine, which is more or less to be expected when one considers the treatment to which the wood is subjected in the creosoting process. (See in this connection the conclusion of W. K. Hatt in Circular 39, Forest Service Experiments on the Strength of Treated Timbers.)

The water absorption is not diminished or appreciably affected by the creosoting process. On prolonged immersion the following results were obtained:

	Water Absorption Per Cent — Untreated Pine	Creosoted Pine	Creosoted Pine	White Oak
3 days' immersion	21.5	23.0	27.0	15.2
7 days' immersion	28.8	26.6	34.1	20.4

These absorption tests on the untreated pine and creosoted pine were made on samples of presumably selected materials which were submitted by bidders on a construction contract.

The samples of creosoted pine from which specimens for test were selected represented contract deliveries of

material for repair work and bidders' samples for construction contracts. The untreated pine blocks were bidders' samples submitted at the same time with the creosoted pine blocks for construction contracts. The samples of white oak represented deliveries of blocks for experimental purposes and for repair work in 1919. The samples of the other hardwoods were sent in as sections of logs from which pieces of the dimensions of paving blocks were cut.

There is nothing new in the use of hard wood for paving blocks. Hard wood pavements have been used successfully in Australia, England, Scotland, and on the continent of Europe. In Australia, karri, jarrah, blackbutt, tallow wood, mahogany and many other varieties of native hard woods have been used. In England and Scotland karri, jarrah and oak have been used. In Australia, England and Scotland hard wood has been used for straight paving from curb to curb, while on the continent it has been used chiefly for special work such as paving in the street railway area and bridge approaches. The Australian hard wood pavements are noted for their endurance, their life having been estimated at 21 years. It is significant that the only disadvantage attributed to pavements of dense hard woods is the tendency of the blocks to wear round at the edges.

In the United States there has been no systematic trial or use of the dense hard woods for paving purposes. White oak in the form of a wedge-shaped block has been laid between alternating rows of granite block in the franchise area of one of the traction companies in New York City. This type of construction consisting of about one-half white oak surface has been used for over 6 years in the railway area at intersections subjected to particularly heavy traffic.

The greater strength and relative insusceptibility to expansion from water absorption of the dense hard woods, such as white oak, chestnut oak and hickory would seem to merit a more extended trial for paving purposes where a wood block pavement is desired.

Earthwork Balancing on Road Work

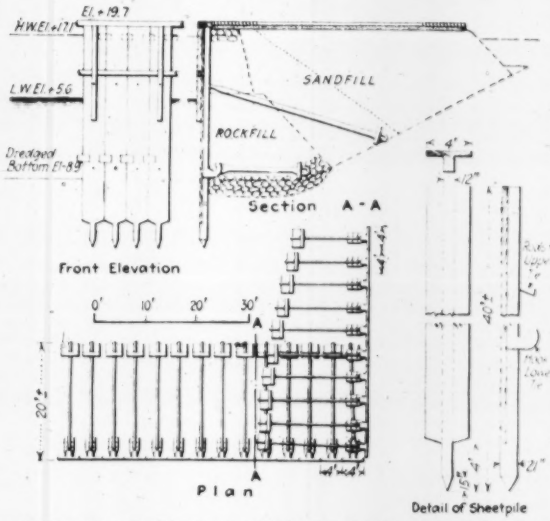
In order to insure sufficient material being provided in a given section of road to make the embankment in that section without waste, the Missouri State Highway Department has issued rules for balancing earthwork which rules also cover shrinkage allowance. It is provided that earthwork is to be balanced to show the disposal of excavation and the method of obtaining material for embankment and to assist in determining economical grade line. As a free haul of 1,000 ft. is specified, the earthwork should balance in sections of not over 1,000 ft. and as much less distance as is practicable. Steepness, irregular profile and fixed points of the grade line may require the sections to be over 1,000 ft. Careful study should be made to determine when it is more economical to borrow, waste or overhaul and at the same time secure a good grade line. The rule for shrinkage is as follows:

Since earth when placed in embankment becomes more dense, occupying less space than it did originally, and since most highway earthwork contains a great amount of vegetable matter, the yardage of excavation should exceed the yardage of embankment. Where excavation averages 100 cu.yd. or more per station, allow 15 to 20 per cent of excess cut over fill. Where excavation averages less than 100 cu.yd. per station allow 20 to 25 per cent excess of cut over fill. Swell should be allowed on rock.

Special Quay Wall for Shores Which Require Filling

Precast Concrete Piles Form Face Wall With Precast Concrete Ties to Hold Them Against Shore Fill

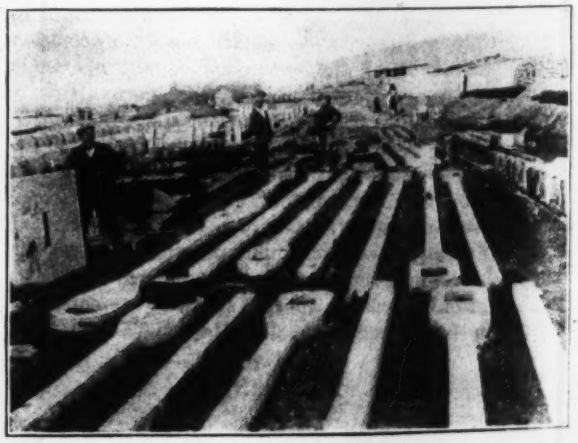
PRECAST reinforced-concrete sheetpiles with specially designed hooked ties are the basis of a new type of quay wall which has been devised by a French engineer, Louis Ravier, of Paris, which has been used in its latest development in the construction of a quay at Kenitra, in French Morocco. The basic idea of the system is the use of a T-shaped sheetpile which is



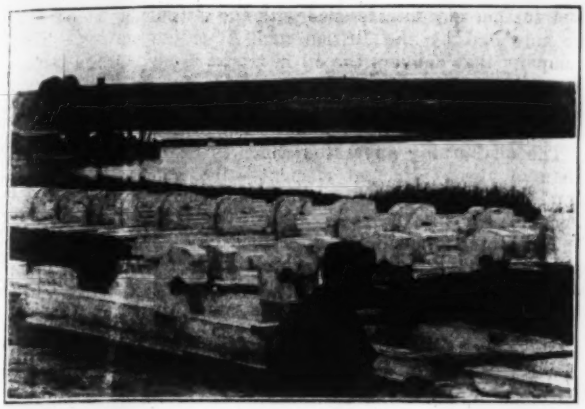
DETAILS OF SPECIAL PRECAST PILE AND TIE QUAY WALL AT KENITRA, MOROCCO, DEVISED BY LOUIS RAVIER

driven in the water and which is held in place against the subsequently placed fill by special ties, the lower one of which is also placed under water before filling is commenced. Afterward the fill is carried from the line of sheetpiles back to the old shore line, and is paved for a solid quay wall.

The T-shaped sheetpile has a main stem 12 x 17 in. in section and in the Morocco wall about 40 ft. long. To this is joined integrally a 4-in. faceplate 4 ft. wide, a thorough connection being made in the reinforcement. This faceplate, however, stops about 4 ft.



VIEW OF THE TIES IN THE CASTING YARD



SHEETPILES WITH HOOKS IN CASTING YARD

above the point of the main shaft of the pile. At a distance of about 12 ft. above this point there is cast integral with the main shaft a heavy concrete hook extending backward from the face of the pile, and at a point about 12 ft. from the top of the pile reinforcing rods are left protruding in the back of the shaft for the future connection of the upper tie rods.

The lower ties consist of precast concrete sections about 12 in. square and 12 ft. long, with eye ends, as shown in the view. The upper tie rods are of not quite as large sections, but are longer and have an eye only at one end, the upper end being left with protruding rods to connect with the rods already left in the sheetpiles.

In construction the sheetpiles are driven in line

CASTING THE SHEETPILES WITH THE HOOKS

through the water to the proper depth, which is not necessarily refusal but is governed by the required elevation of the quay and the pre-ascertained condition of the soil in which the piles are driven. This height is such that the lower hook is just above the bottom of the harbor. The lower tie is then placed under water engaging the hook on the sheetpile by means of a solid concrete cylinder which passes through the eye of the tie and the curve of the hook. The rear end of the tie is held by a reinforced-concrete plate about 4 ft. square kept in place by a concrete block threaded through the rear eye of the tie.

A filling of rock is then dumped into the water to water level, at which height the upper tie rod is placed, in open air, the tie to the sheetpile being made by joining the reinforcing together and pouring the concrete around it. The lower end of the upper tie has a deadman similar to that used in the lower tie. Rock fill is then continued as shown in the cross-section to the top of the quay and the final fill made with sand. The sheetpile is capped with a concrete cap and the quay wall paved as shown.

Progress on Proposed American Falls Dam

ONE of the irrigation projects visited by the Appropriation Committee of Congress this summer and the one seemingly the most likely to be developed soon is the American Falls Dam in Idaho on the Snake River. It would have a storage capacity of 3,000,0000 acre-ft. and would be capable of supplying 1,300,000 acres already irrigated with a safe and reliable supply. In addition it would take care of new land amounting to 400,000 or 450,000 acres.

Prof. W. O. Crosby reported Jan 15, 1920, favorable geological conditions. I. W. McConnell, consulting engineer, New York, headed a board of engineers, selected jointly by the U. S. Reclamation Service and the State of Idaho, and reported April 10, 1920, favorably on the construction of the reservoir. In the fall of 1919 a mass meeting to inquire into the storage needs of the various canal companies brought application for 550,000 acre-ft. of storage. These various interests through a storage committee negotiated with the Reclamation Service for a form of contract providing for Federal construction of the dam with the understanding that the various organizations desiring storage would advance the money for their proportionate share of this reservoir, while Congress would be asked to appropriate money to provide for the storage for lands held in public ownership. Contracts have been signed for 335,170 acre-ft. of storage and the first payment of money has been deposited with the Reclamation Service. The signed contracts are with 31 canals and individuals representing 500,000 acres. Other lands already in cultivation need 550,000 acre-ft. additional. The North Side Pumping Unit of the Minidoka Project (U. S.R.S.) will require 520,000 acre-ft., making a total of 1,400,000 acre-ft. for which there is immediate need.

A 43-ft. dam holding 600,000 acre-ft. can be built for $8.83 per acre-ft., a 64-ft. dam with a capacity of 1,500,000 acre-ft. for $5.73 and the full capacity, 3,000,000 acre-ft. can be obtained with an 89-ft. dam 5,000 ft. long at a cost of $4.50 per acre-ft. of storage. For the full development Congress would have to appropriate $9,370,000 and the land owners $3,090,000. For the 1921 expenditures Congress is asked for $370,000 and the land owners will supply $670,000.

Ohio's 1919 Typhoid Rate Lowest of Record

In 1919 Ohio had a typhoid fever death rate of 8.6 per 100,000 population, the lowest of record. In 1918 it was 14.6, a figure about the average for the three years previous to 1918.

Increasing the Efficiency of Three Pumping Plants

Water Department of Lynn, Mass., Rebuilds Two Low-Lift Plants and Changes from Coal to Oil at Main Pumping Station

RADICAL CHANGES in three pumping stations of the water-works of Lynn, Mass., have been made within the past two years to keep abreast of the times and to get a proper plant efficiency, said Reeves J. Newsom, commissioner of water, in a paper read before the New England Water Works Association at its recent convention in Holyoke. As a result two outlying low-lift stations now have complete equipments of motor-driven pumps of much increased efficiency that may be operated "24 hours a day for weeks at a time practically without attendance," while the main pumping station has had its steam boilers converted for the use of oil instead of coal for fuel, at great reduction in fuel cost.

One of the outlying stations had been equipped with a 15,000,000-gal. centrifugal pump no longer ago than 1912. All this equipment went to the junk heap, including a priming pump of the ordinary reciprocating type. This pumping plant was operated six years without knowledge of its efficiency. In 1918 Mr. Newsom found by tests that when pumping from 12,750,000 to 16,750,000 gal. a day under heads from 10.9 to 27 ft. the efficiency of the combined plant varied from 26 to 47 per cent. The acceptance test of the new plant showed an efficiency of 74.5 per cent. At both of the new outlying stations the priming pumps, Mr. Newsom states, "are novel adaptations of a direct-connected motor-driven hydro-turbine vacuum pump and, complete, occupy a space only 20 x 40 in.

Oil Substituted for Coal

At the main station, where oil has been substituted for coal as fuel, the steam-driven pumps have their steam supplied from two 175-hp. boilers. The reasons for changing to oil, the nature of the contract for oil, the oil equipment and a summary of the advantages of using oil at Lynn are set forth by Mr. Newsom as follows:

The coal problem has become very serious in two ways. At times coal is scarcely obtainable and last winter for several weeks we had to depend on trucks coming through the deep snows from a city twenty miles distant to keep our pumps going. Then, too, the quality of the coal now on the market has made its use very uneconomical. The station duty has dropped as much as 30 per cent at times, and in order to keep up steam there has had to be wasted unburned, through the ash pit, 18 to 20 per cent on the average, and at times as high as 28 per cent of the coal fired. Combined with these facts has been the ever rising price of coal from around $4 per ton to $16.50 at the present time.

We have made a contract for oil at the equivalent of about $9 per ton for coal, and the price is guaranteed for two and one-half years, and the delivery of the oil for five years. This contract is backed by a $10,000 bond, which is two-thirds of the cost of the oil burning apparatus, and the amount to be saved is such that if the oil company delivers oil for only a few months we can change back to coal without loss.

The oil situation appears to be pretty stable, however, when it is remembered that enormous royalties are paid to the Mexican government on the output of oil and it is therefore vitally interested in keeping them in operation, and further any interference with the oil output would not be tolerated by the British and American navies. The company that delivers the oil in trucks from storage tanks in Chelsea, Mass., owns also the wells, the pipe lines, and the tank steamers which bring it to this country, so that transportation difficulties would seem to be minimized.

The oil burning apparatus which is being installed consists of three principal elements—the storage tank, the combined pump and heater, and the burners, with connecting piping and auxiliaries.

The storage tank is of reinforced concrete, built in two separate compartments with a total capacity of 35,000 gal., or about three week's supply. A suction pipe comes from each compartment of the tank and runs to the pump inside the boiler room. These pipes are surrounded near the end by steam jackets which heat the heavy fuel oil so that it will flow. Pipes extending to the bottom enter the tank at the same points to which ejectors can be attached for removing water which may collect from time to time.

Equipment Details

The pump to which the suction pipes are attached is of the double duplex direct-acting type, mounted above the heater, which is cylindrical in shape, the whole thing being a small compact unit. The heater is constructed like a surface condenser, the steam being inside the tubes and the oil flowing around them. In this heater the temperature of the oil is raised to about 130 deg. F.

From the heater the oil is pumped to the boiler front, where it passes through an auxiliary heater composed of another steam jacketed section of pipe which is used to heat the oil beyond the pump when the boilers have been banked, or when for any reason the main heater does not function properly. The piping is so arranged that all exhaust steam from heaters and pump returns to the boilers.

The oil then passes through a regulator and to the burners, where it is atomized by steam and mixed with air. The burners are placed just below the location of the coal grates, the pipes coming in through the ash doors which are entirely bricked up except for the requisite air slots.

The regulator is actuated by changes in steam pressure and controls the flow of oil to the burners. The supply of air is controlled by the position of the chimney drafts. It is possible to obtain a regulator which will also control the air, but a centrifugal pump load is so steady that practically no change is required in the amount of air needed, once it has been set to meet the atmospheric conditions for the day's run, and the complication of such a regulator is not, therefore, justified in our installation.

We use both boilers ordinarily to carry the load and to prevent its unequal distribution and the overloading of either of the boilers. Steam flow meters are being installed to show the respective outputs.

The dangers in the use of oil are two—having too hot a fire, and having the fire too concentrated. The limit to which an oil fire can be forced is usually beyond the safety point of the boiler, and so the output must be watched. If the burners are too close to the boilers the flame may be so concentrated that the rivets in the shell will melt. A boiler setting built especially for oil is usually very high, but a coal installation can in most cases be adapted to oil by removing the grates and putting the burners in the top of the ash pits.

The principal advantages which will in our case be derived from the use of oil are as follows: (1) Oil is cheaper than coal; (2) it can be burned more efficiently than coal; (3) greater boiler capacity can be developed; (4) coal and ash handling charges will be eliminated; (5) variation in quality will be minimized; (6) banking of fires can be done very much more economically; (7) neater and cleaner and better working conditions will be obtained.

The burning of oil seems to solve our problem in this station, temporarily at least, and perhaps until such time as the eventual solution, the development of available water power will be consummated.

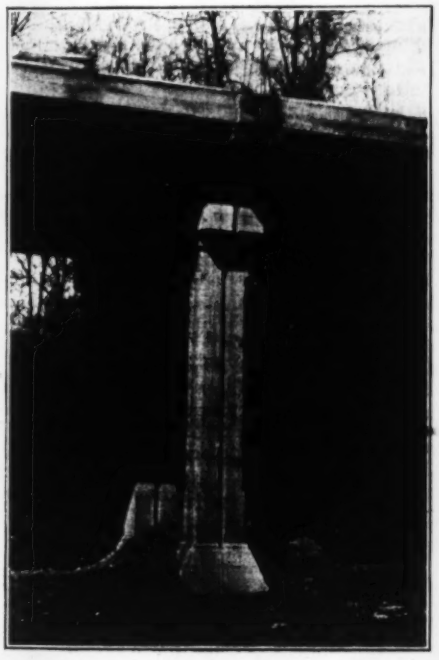
Concrete Abutment of Thin Walls Tied by Diaphragm

Special Design Used in Illinois Highway Viaduct Saves Material and Insures Coaction of Wings and Face Wall

FOR the abutments of the New Athens Viaduct, a concrete beam-and-slab highway bridge for St. Clair County, Illinois, a special type of wing-wall abutment was designed and used by A. P. Poirot, of Bellville, Ill., who built the bridge. Comparatively thin wing and face walls were used, but were tied together near the top by a horizontal diphragm spanning the juncture angle.

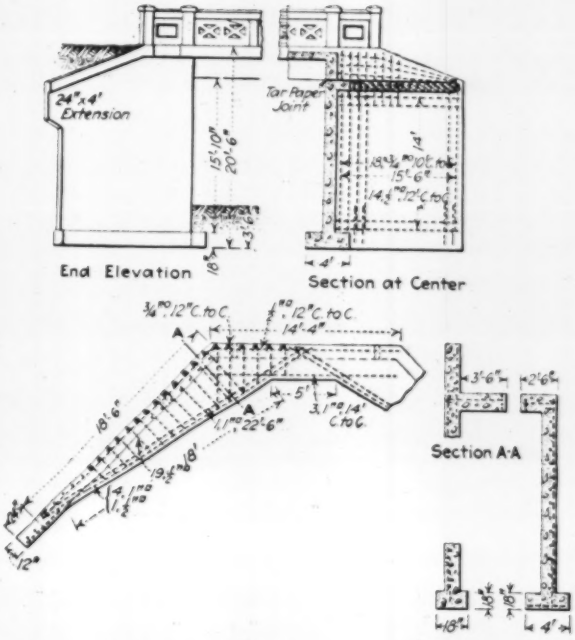
The viaduct consist of 20 spans of 40 ft. and two spans of 37 ft. 9½ in. The roadway, 19 ft. wide, is carried on a 7-in. slab with three longitudinal 16 x 26 in. stringers, which are supported on three-column bents. Except that every other bent is of the split-column type for expansion, as shown in the view herewith, the design follows ordinary practice.

A detail and a view of the wing-wall abutment are also shown. The abutment consists of a face wall 14 ft. 4 in. long and 20 ft. 6 in. high, which is 12 in. thick in its main section, with a 4 ft. base and a 2 ft. 6 in. corbeled top. Joined to this at a 45 deg. angle are 18 ft. 6 in. wings also 12 in. thick but extending up higher, at the face, than the face wall. Joining the three walls at the elevation of the face-wall corbel is the 16 in. horizontal diaphragm, of the shape and reinforcement shown in drawing. The advantages claimed for this type of construction by the designer are as follows:



SPLIT EXPANSION COLUMN BENT ON NEW ATHENS VIADUCT

Details of diaphragm abutment of New Athens viaduct, Clair County, Illinois.



DETAILS OF DIAPHRAGM ABUTMENT OF NEW ATHENS VIADUCT, ST. CLAIR COUNTY, ILLINOIS

(1) It permits the use of comparatively small sections for the wings and face wall and no footings need be provided under the wings further than those required to carry the deadload of the wings themselves.

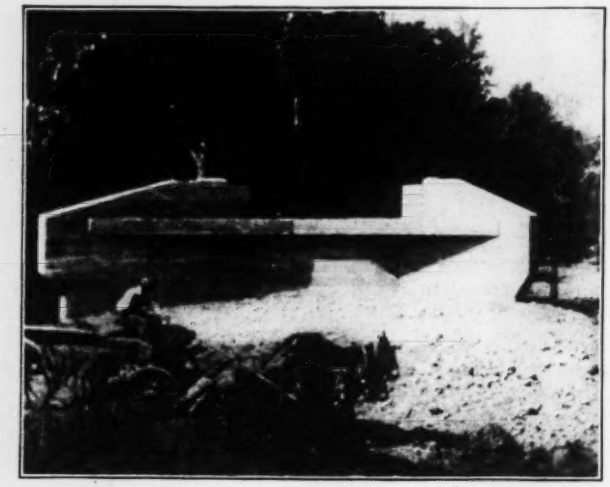
(2) The location of the diaphragm is such that it can be built strictly according to the plans with ease. The steel can be properly placed and the concrete properly formed and worked, thus assuring good construction at the critical part. This is often not the case in other types of design. For instance, in the cantilever type the critical part of the wings is usually below the stream bed immediately above the footing, at a point where form construction for the wing wall makes it almost impossible to keep a thin layer of dirt from separating the wall from the footing course, thus providing a chance for corrosion of the rods where it is least desired, and consequently a lessening of the value of the structure.

(3) The base can not be broken when back filling is being done, as is often observed when ties are used that extend underneath the roadway from the end of one wing to the end of the other.

The wing walls and the face wall are designed, so far as the earth pressure in back of them is concerned, as slabs supported at the top and bottom. The earth in front of the footings and the friction on the earth underneath provide the bottom support, and the diaphragm brace, shown above, provides the means whereby the wings and the wall mutually support each other at the top.

The base is designed to resist bending moment at any section caused by the tendency of the wings to spread and also to resist the moment caused by the direct dead- and live-load from the top.

As a test, the back filling of one abutment was made shortly after it was completed and before the superstructure was placed. This remained for about eight



INSIDE OF WING-WALL ABUTMENT WITH CONNECTING DIAPHRAGM

months, after which the last span was put in place. Careful measurements revealed no movement and no signs of insufficient strength were found.

D. O. Thomas is county superintendent of highways of St. Clair County and Ben A. Gundland was in charge as superintendent of construction.

Methods of Increasing Efficiency of Track Labor

Cost Records Necessary—Permanent Employment With Good Pay and Housing Help To Secure Better Men and Better Work

UNDER present conditions of labor and wages the railways have a very practical interest in increasing the efficiency and economy of track work. New methods and better management will be necessary to obtain this result. The situation was reviewed by F. G. Jonah, chief engineer of the St. Louis-San Francisco Ry., in an address delivered at the recent annual meeting of the Roadmasters and Maintenance of Way Association, an abstract of which follows:

Formerly, when labor for track work was abundant and cheap, there was little check on its performance or economic results. In considering the efficiency of workmen the first thing necessary is to know accurately what they are doing now and what they are capable of doing. This means putting the work on the basis of unit costs. In the past, there have been records only of the total expenditure for various subdivisions of track work, comparatively little being known of the amount of work performed for this expenditure. In the transportation department the whole efficiency of the organization is judged by statistical information, based on the unit cost of moving a ton of freight one mile or carrying a passenger one mile; the mileage of engines per ton of coal; the movement of loaded cars per day, the terminal expense of handling cars and engines. The information shows at a glance whether the railway's business is being handled economically.

It is possible to measure in the same way results of the work of track gangs, so that various railroads are beginning to undertake the collection of such data. The system introduced on the St. Louis-San Francisco Ry. is found to be well worth its cost, and is giving some definite results. The basis of this system is the time book. Heretofore, track foremen have reported the amount of time put in on different kinds of work, but have said nothing about the quantity of work done. The new time book provides for such a record. When the cost of work is known, steps can be taken to keep up a good average or to raise a low one.

Motor cars for section forces were introduced by many railways a few years ago, but on the assumption that a man could then cover more territory, they lengthened the sections and eliminated a few foremen, apparently trying to figure sufficient saving in wages to pay for the motor cars. This was at a time when men worked ten hours per day. With the eight-hour day now general, the long section is a mistake, as too large a percentage of time is taken up in moving to and from work. Further, increased traffic, with increased number and weight of trains, demands more intensive work and supervision, so that many sections should be shortened, instead of lengthened. But the motor car will be economical even on short sections. Power appliances should be used whenever possible and manual labor reduced to a minimum.

With increased wages, a better personnel can be secured. All sections should be allowed to carry a permanent force of at least a foreman and four men, expanding to a greater number at times in the summer. Steady employment the year round will hold good men on the sections, and give the railway a chance to develop good foremen for section gangs and extra gangs. Consideration should be given to proper housing facilities. The time for box car bodies as houses for families is past, and the day of the hobo is past. Clean, efficient labor demands comfortable, commodious, healthful domiciles. It will pay to establish these. There is no reason why the section foreman should not be one of the substantial citizens of the community in which he lives. His pay now compares favorably with that of the village school teacher, and the railways will expect their men to measure up to a higher standard than has prevailed in the past.

Extra gangs should be housed in clean bunk cars and the train should carry a car equipped with shower bath. Cars should be screened and a good commissary maintained. In the days of the hobo camp, few could blame the man who put in a month at one and then took the first opportunity (at pay day) of going off on a drunk to forget his misery. He drifted back to camp only because he had no other place to go. Labor cannot be held under conditions that prevailed in the past. The road that will get the best results is the one that provides the best housing conditions and has the best personnel; and the latter depends largely upon the former.

After good men have been secured, they must be fully equipped with tools, so that they can undertake any kind of work at any time. Ample material must be supplied also, for money is wasted if extra gangs and section gangs are kept waiting for material. It is unwise to undertake too much work by doubling up section gangs, for the morale of a section foreman is destroyed by frequently taking him off his own territory to help another foreman. The practice should be resorted to only in such emergencies as wrecks and washouts, for it is far better to organize extra gangs to do extra work. If three or four section gangs are worked together, at the present pay of section foremen, the overhead expense is much more than the salary of a good extra-gang foreman.

Dallas Has Aërial Photo Map

Dallas, Texas, is one of the few cities in the country that possess a complete aerial photographic map. It was made last spring by army aviators at the request of Mayor Wozencraft and now hangs in the Mayor's office. Three hundred seventy-eight plates were exposed at an altitude of 6,000 ft. from a former bombing plane flying along parallels. Later these pictures had to be cut and carefully fitted together, making a mosaic 5 x 8 ft. in size. This map will be of service in planning Greater Dallas.

Am. Soc. C. E. Arguments for and Against Federation

Membership Receives Presentation of Opposing Views Prior to Voting on Question of Joining the New Organization

IN accordance with the action of the Board of Direction of the American Society of Civil Engineers at a meeting held in Portland, Ore., Aug. 12, there has been sent to the corporate membership of the society ballots calling for a vote on the question of joining the Federated American Engineering Societies, together with a pamphlet containing arguments pro and con, prepared by committees of three consisting of Edward E. Wall, David C. Henny and John C. Hoyt, who present the case in favor of joining the federation, and George H. Pegram, John W. Alvord and Arthur S. Tuttle, who oppose it. The arguments are reprinted in full below:

Arguments for Joining Federation

As Prepared by Edward E. Wall, David C. Henny, and John C. Hoyt.

ON JUNE 18, 1918, the Board of Direction of the American Society of Civil Engineers adopted resolutions creating a Committee on Development. About the same time similar committees were created by the American Institute of Mining and Metallurgical Engineers, the American Society of Mechanical Engineers, and the American Institute of Electrical Engineers. In order to correlate the work of these committees so far as it pertained to matters of common interest to the engineering profession, especially to matters of public welfare that involved engineering training and technical experience, these committees appointed from their membership a general committee, known as the Joint Conference Committee, consisting of fourteen members. Three members of this committee were also members of the Engineering Council.

The Joint Conference Committee, after considering various plans for providing an agency through which the engineers of the country could speak and act as a unit for the engineering and allied professions, was unanimously of the opinion that to establish such an agency some comprehensive organization other than Engineering Council was necessary—that an organization was needed which should be thoroughly representative and democratic and which should be formed from the bottom up and not from the top down. This committee submitted a unanimous report which contained a general plan for such an organization based on the following principles:

1.—Non-interference with interrelations of organizations with respect to technical matters, and maintenance of the autonomy, functions, and operations of each organization.

2.—Local affiliation of existing groups of engineers in order to facilitate united action on local questions of public welfare and other matters of common interest.

3.—National association of engineering organizations by means of a national council composed of representatives widely chosen by local affiliations or organizations and by national organizations, the council to meet annually and to act through an executive board.

4.—Financial support of such an association by contribution from all participating organizations on a basis of membership.

5.—A form of organization which will permit expansion and development.

The report of the Joint Conference Committee was considered by Engineering Council at its meeting in October, 1919, and the general principles therein stated were endorsed. The report was also presented to the four Founder Societies and it was considered at a joint meeting of the governing boards of these societies held Jan. 23, 1920, together with the governing boards of the American Society for Testing Materials and of the United Engineering Society and with members of Engineering Council. At this meeting resolutions were unanimously adopted requesting the Joint Conference Committee to call, without delay, a conference of national, local, state, and regional engineering organizations and affiliations for the purpose of forming the comprehensive organization of engineers recommended. In compliance with this request a call was issued to about 110 engineering and allied technical societies not organized for commercial purposes, but devoted chiefly to the advancement of engineering and allied technical arts, to send representatives to an organizing conference to be held in Washington, D. C., June 3-4, 1920. There were in attendance at this conference 140 men, representing 71 societies having an aggregate membership of over 110,000, or 80 per cent of the membership of the organizations which were invited to send representatives to the conference.

The conference was, therefore, in every way thoroughly representative, the delegates coming from all parts of the United States and representing national, local, state, and regional engineering organizations and affiliations. Its outstanding feature was the unanimity of opinion that a comprehensive body to speak for allied and technical organizations was necessary. Without a single dissenting vote the Federated American Engineering Societies, the management of which shall be vested in a body known as the American Engineering Council, was formed. A constitution and by-laws were adopted (see accompanying copy) and the Joint Conference Committee was instructed to act as the ad interim committee between the adjournment of the conference and the convening of the American Engineering Council on Nov. 10, 1920.

Engineering Council, at its meeting on June 17, 1920, unanimously endorsed the Federated American Engineering Societies and its governing body, the American Engineering Council, and authorized "its executive committee to offer such assistance as may be practicable in completing the work of the Organizing Conference of the Joint Conference Committee of the Founder Societies in establishing the American Engineering Council."

The chairman of Engineering Council, under date of July 12, 1920, stated:

"I am sure that, as chairman, I voice the opinion of Engineering Council when I state it will do everything in its power to advance the organization of the new council, to act in the interim, and to facilitate a smooth transfer of activities from the old to the new council."

From this it is quite evident that Engineering Council will go out of existence when American Engineering Council is prepared to take up its work, about Jan. 1, 1921.

The Federated American Engineering Societies has now a membership consisting of the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, the Detroit Engineering Society, the Dallas Technical Club, and the Cleveland Engineering Society. The sentiment expressed by delegates at the organizing conference indicates that other societies to whom invitations to become members have been extended will be added to the list as soon as the question can be considered by their governing boards and presented to their membership, so that without a doubt the Federated American Engineering Societies when completely formed will speak for a body of over 100,000 engineers and allied technologists and will be prepared to perform a real function as a medium of the engineering and allied technical professions in their relation to the public.

The new organization is no mushroom. It has not grown in a night. It is not a dream of one or of a few men. It is the mature result of months of hard thought and days of intense conference by many broad-minded engineers from all quarters of our land, gathered day after day.

The following events indicate that now is the psycho-

logical time for establishing a comprehensive organization to represent engineering and allied technical interests, and that this new organization is to be the Federated American Engineering Societies with a governing body to be known as the American Engineering Council.

1.—On Sept. 17, 1919, the Joint Conference Committee of the four Founder Societies unanimously recommended this comprehensive organization.

2.—In October, 1919, Engineering Council unanimously endorsed this recommendation in principle.

3.—On Jan. 23, 1920, the governing boards of the Founder Societies, of the American Society for Testing Materials, and of the United Engineering Society and the members of the Engineering Council unanimously endorsed the recommendation of the Joint Conference Committee in principle, and requested the committee to call, without delay, a conference of representatives of national, local, state, and regional organizations and affiliations, for bringing into existence the comprehensive organization recommended by the Joint Conference Committee.

4.—This conference met in Washington, June 3-4, 1920, and unanimously voted in favor of the comprehensive organization recommended by the Joint Conference Committee.

5.—Engineering Council, at its meeting on June 17, 1920, unanimously endorsed the formation of the Federated American Engineering Societies and the American Engineering Council and instructed its executive committee to proffer and perform every possible service in bringing it into existence.

6.—The technical papers of the country have been unanimous in their endorsement of the action of the Washington Organizing Conference and in their expressions that the step taken was the most important in the history of the engineering profession.

The American Society of Civil Engineers recognized the need of a general organization to represent the engineering and allied technical professions when it joined in the creation of Engineering Council. The American Engineering Council is a similar organization made more democratic by including in its membership local as well as national societies, and it is obvious that any organization representing many societies in different parts of the country must have greater potential effect that one representing only a few.

The American Society of Civil Engineers has given financial support and in every way fostered Engineering Council; and there is no reason why it should not lend the same support to the American Engineering Council, which will succeed Engineering Council and carry on its work in a more representative way and on a more comprehensive scale.

The statement has been made that the proposed plan for financing the American Engineering Council will not provide adequate funds. Engineering Council has done a large amount of work under appropriations from its five member societies, never exceeding $22,000 per annum. It has, of course, always had need of more money, but that is a need of any live organization. The proposed budget for the American Engineering Council is several times that heretofore allotted for Engineering Council and is considered ample to provide both for its successful organization and for carrying on a large amount of useful work.

One of the fundamental principles of the Federated American Engineering Societies is "non-interference with the interrelations with respect to technical matters and the maintenance of the autonomy, functions, and operations of individual organization"; therefore, the American Society of Civil Engineers will lose nothing by becoming a charter member of the Federated American Engineering Societies, and its traditions and achievements, together with its high standing in the engineering profession, will be strengthened through its hearty support of the movement. In fact, the American Society of Civil Engineers cannot afford to refuse to be identified with the movement.

On April 14, 1920, the questionnaire ballots of the members of the American Society of Civil Engineers on the report of the development committee were canvassed, with the following results:

Question A-1: "Shall the American Society of Civil Engineers 'adopt the principle of becoming an active national force in economic, industrial, and civic affairs'?" Carried by a vote of 2,889 to 910, the percentage of affirmatives votes being 76 per cent of those who voted on the question.

Question A-2: "Shall the society actively co-operate with other engineering and allied technical associations in promoting the welfare of the engineering profession?" Carried by a vote of 3,115 to 685, the corresponding percentage of affirmatives being 82 per cent.

It is evident that the sentiment of the society is strongly in favor of these policies, but it was also evident that at that time many members felt that it would be unwise for the society to join any federation which did not grant the national organizations the same representation accorded to local, state or regional organizations. At the organizing conference of the Federation of Engineering Societies, held at Washington, D. C., June 3-4, 1920, this defect in the original plan was remedied and this action, it is believed, removed the only objection that can be consistently urged as a reason why the American Society of Civil Engineers should not join the new federation.

The question now before the membership of the American Society of Civil Engineers is whether the society shall join this organization in its formative state and thus be in a position to play an active part in developing its policies, or whether it shall lag behind and permit other organizations to perform this work, remaining aloof until others have done the construction and then apply for membership.

It is pertinent to refer to the parallel case illustrated by the United Engineering Societies Building on West 39th St., New York City. Every member of some years' standing will recall that at the time of Mr. Carnegie's gift the American Society of Civil Engineers had an opportunity to enter the engineering building along with the other national societies, and that, owing to the influence of the ultra-conservative element in the society, it did not avail itself of this opportunity; after several years elapsed wiser counsel prevailed and the society took the step which it should have taken in the first place. Shall the members of the society be contented to repeat a mistake of this kind?

The American Society of Civil Engineers should be a leader in this movement, and the vote in favor of its becoming a charter member of the Federated American Engineering Societies should be overwhelmingly in the affirmative.

Arguments Against Joining Federation

As Prepared by George H. Pegram, John W. Alvord, and Arthur S. Tuttle.

1. The federation is identical in principle with the federation plan which was disapproved by the society as Question No. 3 of the questionnaire voted on early this year.

The reasons which then demanded its disapproval still obtain.

2. The purposes of the federation are vague and ambiguous. Advocacy of the plan has relied more upon rhetorical persuasion than upon definite and clean-cut statement as to what is to be done and how it is to be accomplished.

3. Because of the vagueness of purpose it is impossible for any one to draw reasonable conclusions as to whether the work of the federation will justify either its creation or the heavy expense it will inevitably entail.

4. The American Society of Civil Engineers is representative and democratic in organization.

The federation is neither representative nor democratic. Its management is vested in a council composed of representatives of constituent societies, who are not necessarily elected by the members of those societies. This council would be in the nature of an autocracy without the usual restraints to the pursuit of political and personal ends.

5. The requirements of membership in the American Society of Civil Engineers demand high personal, technical, and executive qualifications, and are rigidly enforced.

The absence of qualifications for membership in the fed-

eration lets down the bars to almost any society that may incorporate the word "engineer" in its title. This diversity of standards and interests will prevent the federation from becoming in any sense a representative body.

To carry weight, engineering prestige must rest on professional attainment and standing rather than on bulk of numbers. The federation stes up no profesisonal standards whatever. Its one standard is bulk.

6. The representation of the American Society of Civil Engineers in the council and executive board would be in an insignificant minority, careful estimates indicating that on the basis of the Washington conference, this representation would be less than 8 per cent. The society would be simply lending its prestige for exploitation and both the society and the engineering profesison would suffer by such a sacrifice.

7. The American Society of Civil Engineers has never yielded control of its actions or policies to others. The nearest approach is in its connection with the Engineering Council, which, however, can take no action from which the representatives of the society dissent.

Should the society join the federation, it would, in the estimation of the public, cease to function as the leading engineering organization, as it has done for sixty-eight years.

8. The revenues of the society even if its dues are increased as proposed under the amendment about to be voted upon, are not adequate to permit the society to maintain membership in the federation without serious curtailment of its technical activities and the entire abandonment of the proposed plan for strengthening the local sections.

9. The experience of the American Association of Engineers demonstrates the impossibility of doing effective "welfare" work with the funds that will be available under the federation scale of membership fees.

10. "Welfare" work in general is of local or regional interest. It can be handled most effectively by local or regional bodies which would be inspired by a genuine community of interest.

When a "welfare" matter demands broader consideration than can be obtained through local or regional organizations, it can and should be made the subject of the co-operative action of interested organization in which this society should take a leading part, the funds for such purposes being raised by special contribution. This will insure the needful community of interest in each instance, and will avoid the wasteful expense arising from the permanent maintenance of the elaborate and costly overhead of a national federation.

11. If the membership of the society, upon further consideration, elect to stand by their previously expressed determination to remain out of such a one-sided and unrepresentative combination in which there interests and rights have been needlessly sacrificed, the society's prestige will not thereby be diminished, but, on the contrary, will be enhanced.

Public Water Supplies in Quebec

Of 230 towns and villages having 500 population or more in the Province of Quebec, 192 with a total population of 1,383,700 have water-works, but all told there are some 440 separate water supplies in the province, if those are included that serve only a few houses. The foregoing figures are from a paper by T. J. LaFreniere before the recent Montreal Convention of the American Water-works Association. Of the 192 works named, 97 take water from rivers. These works serve a population of 1,192,512, or 86.2 per cent of the total supplied by the 192 plants. Of the remaining works, 20 supply lake water to 3.8 per cent of the total population involved, and 75 supply water from springs and wells to 10 per cent of the population supplied by the 192 works.

Quays vs. Piers for Ports

COMPARING the virtues of the quay system with the pier system for shipping ports, Capt. F. T. Chambers, U. S. N., of the U. S. Shipping Board Port Facilities Commission, in an article in *Engineering News-Record*, Sept. 16, 1920, p. 556, showed the same

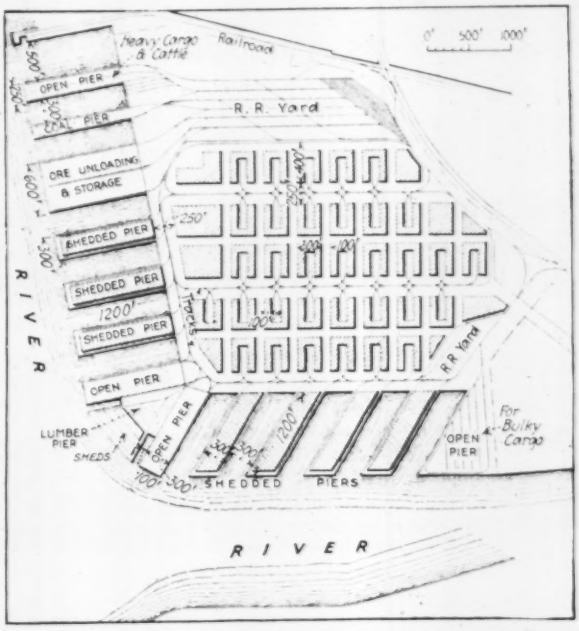


FIG. 1. PIER LAYOUT FOR A PORT AT JUNCTION OF TWO RIVERS

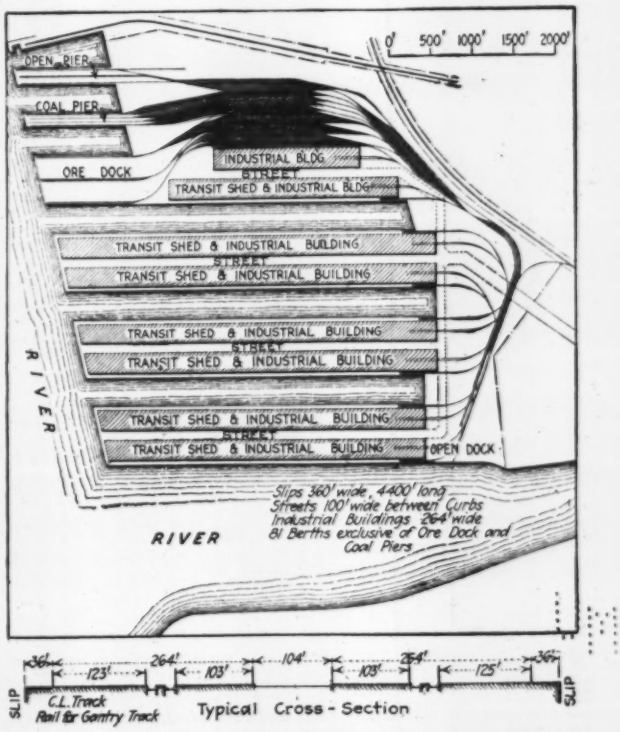


FIG. 2. QUAY LAYOUT FOR SAME AREA AS SHOWN IN FIG. 1

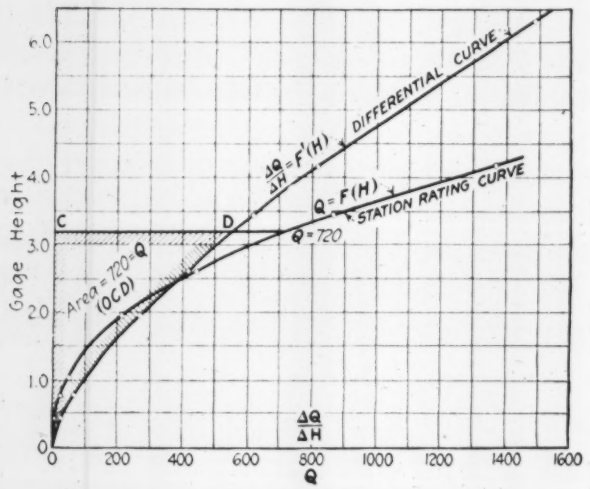
area as developed for the two types. By an error in make up the layout for the quay system was omitted and as Fig. 2 of the article was given the plan of the layout given in cross-section in Fig. 3. There are reproduced herewith, therefore, the two layouts for the same area, as intended in the original article. It is the contention of Captain Chambers that the quay layout, shown as Fig. 2, is much the better from every standpoint.

Differential Method for Drawing Stream Rating Curves

By C. C. Jacob

Federal Court Water Commissioner, Assistant Professor of Civil Engineering, University of Utah, Myton, Utah.

TO OBTAIN a continuous record of the flow of water in a stream or over a weir it is first necessary to obtain the functional relation between the gage height and discharge, or the rating curve. In the case of river gaging stations the ordinary method is to make current meter measurements of discharge at various observed gage heights. For weirs we rely upon the formulas and tables published by various experimenters, selecting the particular formula that best fits the case in hand. Very often, however, the hydraulic engineer must determine the flow of a stream which cannot be directly measured, or the discharge over a dam or weir of such size or shape that no experimental data applicable to the problem are available. The following indirect method was developed by the writer in an attempt to rate a gaging station on a rough mountain stream where no satisfactory measuring section could be found. The accompanying curves and table illustrate the method which was developed theoretically as follows:



STREAM RATING CURVE DRAWN BY DIFFERENTIAL METHOD

ΔQ	ΔH	ΔQ/ΔH	H	Q
5 6	0 20	28	0 42	4 76
10 8	0 18	67	0 76	20 6
15 9	0 15	106	1 04	44 6
37 4	0 14	267	1 95	211
58 0	0 14	414	2 63	442
92 0	0 15	617	3 45	862
122 0	0 15	813	4 16	1,370
179 0	0 12	1,490	6 30	3,810

NOTE—For any gage height, the area *OCD* between the differential curve and the *H* axis is equal to the discharge for that gage height.

Consider a stream or weir whose discharge, Q, cannot be directly measured, on account of unfavorable channel conditions or for any other reason. To apply the differential method it must be possible to discharge into the stream above the gage a comparatively small stream which has been accurately measured. Call this smalll stream, or increment of discharge, ΔQ. It will produce a certain increase, ΔH in the gage height, H, at the main stream gaging station, which can be accurately observed with a hook gage. Several values of ΔQ and ΔH are obtained for various stages of the stream. From these data a curve is plotted with values of $\Delta Q \div \Delta H$ as absissæ and corresponding values of H as ordinates. This is obviously the differential curve of the station rating curve required, expressed as $\Delta Q \div \Delta H = F'(H)$, $Q = F(H)$ being the general expression for the rating curve. The rating curve, being the integral of the curve plotted from the observed data, is obtained from the latter by measuring the successive areas between the curve and the H axis by means of a planimeter or simply by "counting the squares." This is, of course, equivalent to integrating the equation

$$\int_{0}^{Q} \Delta Q = \int_{0}^{H} F'(H)\, \Delta H \text{ or } Q = F(H)$$

Any convenient stream which can be diverted temporarily around the gaging station and accurately measured with a current meter or standard weir can be used as the "increment stream." It must be of sufficient size to give a measureable increase in the station gage height, the error in the rating curve derived by this method being proportional to the error in the observed gage height increment. The "stage of zero flow" should be determined if practicable, but this can be obtained indirectly by taking advantage of the fact that the rating curve for open channel sections as well as weirs has the parabolic form $Q = CH^n$, and the differential curve will give a straight line graph on logarithmic paper.

Figures on Sale of Rhone Power

Some of the transpositions of power costs from French to American currency were erroneously stated in the article "French Government to Regulate River Rhone" by Thorndike Saville, in *Engineering News-Record*, Aug. 26, 1920, p. 396. The last paragraph of the first column on p. 398 should have read as follows:

Under these conditions, if during the first period of ten years it is possible to sell 300 million kw.-hr. at 0.05 fr. (0.96c.) and 330 million kw.-hr. at 0.03 fr. (0.58c.) there will be, after payment of interest on stock and bonds, an income of 1,000,000 fr. ($193,000) during each of the first ten years. During the second period of forty years, in the course of which the bonds would be retired, it will be sufficient to sell 800 million kw.-hr. at 0.04 fr. (0.78c.) and 550 million kw.-hr. at 0.015 fr. (0.29c.) for the annual net income to be two million francs ($386,000). During the third period of twenty-five years, in the course of which the stock will be retired, it would be sufficient to sell 1,000 million kw.-hr. at 0.03 fr. (0.58c.) and 800 million kw.-hr. at 0.01 fr. (0.19c.) to give a net annual income of twenty-one million francs ($4,050,000). At the expiration of the concession it should be possible to sell 1,000 million kw.-hr. at 0.02 fr. (0.39c.) and 800 million kw.-hr. at 0.01 fr. (0.19c.), and there would be an annual income of sixteen million francs ($3,086,000).

Pennsylvania and Rhode Island Traffic Counts Compared

MAIN line highways leading north and south from Providence, R. I., show as much, or greater, passenger-car and commercial motor-truck traffic than at any of nine points along the Lincoln Highway in Pennsylvania, or than at two points on the William Penn Highway, also in Pennsylvania, according to traffic censuses made during July. Traffic counts were begun upon the Rhode Island and Pennsylvania roads upon the same day—July 25. The Rhode Island counts continued during the ensuing week ending Sunday, Aug. 1. The counts in Pennsylvania were taken for July 25 and 26 only. Counts were made in Pennsylvania beginning at 6 a.m. and ending at 6 p.m., while those in Rhode Island were taken during the 18-hr. interval from 6 a.m. to midnight.

Traffic Count Point	Passenger Cars	Trucks
Post Road, Camp Ave., North Kingston	13,723	1,125
Post Road, Matunoc	5,789	448
West Shore Road, Warwick, Conimicut	20,162	3,427
Nooseneck Hill Road, Spring Lake	2,502	376
Plainfield Street, Providence City Line	5,454	2,285
Hartford Pike, Oak Swamp Reservoir	3,893	700
Putnam Pike, Barnes Hill	8,539	1,247
Taunton Ave., West of Pawtucket Ave	13,316	2,003
Waterman Ave., West of Pawtucket Ave	11,070	2,404
Pawtucket Ave., Brightridge School	25,182	4,398
Nayatt Point Road, West Barrington	13,376	1,437
Child St., Warren	10,119	1,733
Warren-Bristol Road, Town Line	7,087	2,238
Fall River to Tiverton, Anthony Hill	13,104	1,765
Sakonett Point Road, Lafayette Road	2,266	490
Main Road, Portsmouth Town Farm	7,944	1,094
New London Turnpike, Emery Park	7,960	1,385

TABLE I. TRAFFIC COUNT ON NINE SECTIONS OF THE LINCOLN HIGHWAY, PENNSYLVANIA—6 A.M. TO 6 P.M.

Point Taken	Kind of Traffic	July 25	July 26
Coatesville-Downingtown	Passenger cars	3,877	2,307
	Motor trucks	82	1,013
	Motorcycles	125	43
	Teams	33	74
	Total	**4,117**	**3,437**
Philadelphia-Morrisville	Passenger cars	2,647	922
	Motor trucks	265	546
	Motorcycles	(a)	.....
	Teams	54	117
	Total	**2,966**	**1,585**
Allegheny Co.—Greensburg	Passenger cars	2,147	920
	Motor trucks	37	188
	Motorcycles	.....	.....
	Teams	1	5
	Total	**2,185**	**1,113**
Greensburg-Westmoreland Co.	Passenger cars	2,027	644
	Motor trucks	32	103
	Motorcycles	149	33
	Teams	22	20
	Total	**2,230**	**800**
Bedford-Fulton Co.	Passenger cars	690	285
	Motor trucks	51	101
	Motorcycles	28	11
	Teams	17	47
	Total	**786**	**444**
Bedford-Somerset Co.	Passenger cars	845	475
	Motor trucks	29	19
	Motorcycles	41	13
	Teams	6	...
	Total	**921**	**507**
Columbia-Lancaster	Passenger cars	1,722	460
	Motor trucks	44	115
	Motorcycles	76	22
	Teams	22	26
	Total	**1,864**	**623**
Lancaster-Chester Co.	Passenger cars	1,356	560
	Motor trucks	31	57
	Motorcycles	70	32
	Teams	34	13
	Total	**1,491**	**662**
Philadelphia-Paoli	Passenger cars	1,303	507
	Motor trucks	33	226
	Motorcycles	79	42
	Teams	11	9
	Total	**1,426**	**784**

(a) Number motorcycles for the two days—343.

TABLE II. TRAFFIC COUNT ON TWO SECTIONS OF THE WILLIAM PENN HIGHWAY, PENNSYLVANIA—6 A.M. TO 6 P.M.

Point Taken	Kind of Vehicle	July 25	July 26
Bethlehem-Easton	Passenger car	3,243	1,002
	Motor truck	75	270
	Motorcycle	159	60
	Team	8	25
	Total	**3,485**	**1,357**
Lebanon-Reading	Passenger car	1,431	431
	Motor truck	34	105
	Motorcycle	148	23
	Team	30	73
	Total	**1,643**	**632**

On the Post Road leading into Providence from the south, account made at Norwood showed a total number of vehicles—automobiles, motorcycles, auto trucks and heavy and light wagons—of 8,141 on July 25. The total number of vehicles on Monday, July 26, decreased to 3,902, and the number of auto trucks increased from 293 on Sunday to 583 the following day. Upon the Mendon Road leading into Providence from the north, a total of 4,542 vehicles was counted July 25. This number decreased to 2,864 on Monday, July 26, though the number of trucks jumped from 151 on July 25 to 671 on Monday, July 26.

TABLE III. COUNTS ON MAIN LINES LEADING NORTH AND SOUTH FROM PROVIDENCE, R. I., 6 A.M. TO 12 P.M.

Point Taken	Post Road—Austins Corners—Norwood								
	Sun. July 25	Mon. July 26	Tues. July 27	Wed. July 28	Thur. July 29	Fri. July 30	Sat. July 31	Sun. Aug. 1	Weekly Totals Sundays, Averaged
Automobiles	7,522	3,129	3,207	3,689	3,687	3,331	4,114	6,274	28,055
Motorcycles	297	140	117	126	117	114	193	237	1,074
Auto trucks	293	583	620	628	710	840	508	331	4,201
Heavy wagons	20	22	13	14	16	14	21	1	111
Light wagons	9	28	24	21	19	26	29	24	163
Totals	8,141	3,902	3,981	4,478	4,549	4,325	4,865	6,867	33,604

Point Taken	Mendon Road—Berkeley								
	Sun. July 25	Mon. July 26	Tues. July 27	Wed. July 28	Thur. July 29	Fri. July 30	Sat. July 31	Sun. Aug. 1	Weekly Totals Sundays, Averaged
Automobiles	4,152	2,047	1,872	2,055	2,074	2,036	2,432	4,376	16,780
Motorcycles	200	61	71	33	66	56	100	175	575
Auto trucks	151	671	554	661	602	658	642	253	3,990
Heavy wagons	2	46	22	33	23	21	32	6	181
Light wagons	37	39	43	33	24	25	49	15	239
Totals	4,542	2,864	2,562	2,815	2,789	2,797	3,255	4,725	21,765

The heaviest traffic in Pennsylvania was found to be traveling the Chester County section of the Lincoln Highway between Coatesville and Downingtown. On Sunday, July 26, 3,877 pasenger cars were counted on this thoroughfare, while the number of trucks was 82. The next day, Monday, the number of passenger cars decreased to 2,307, while the truck total jumped to 1,013. The total number of vehicles counted on this section of road on Sunday was 4,117, and on Monday, 3,437. The heaviest traffic on the William Penn Highway was found on the section between Bethlehem and Easton. On Sunday, July 25, 3,485 vehicles traversed this section of the highway, 3,243 being passenger cars and 159 trucks. The following day there was a total of but 1,357 vehicles, 1,002 being passenger cars and 270 being motor trucks.

Inasmuch as an average was taken of the two Sunday counts the figures given above represent but seven days traffic.

In all counts made in both states the week-day passenger vehicle traffic approximated 50 per cent of that on Sunday, while the week-day truck traffic was generally from three to six times that reported on Sunday.

Counts were made at 19 points in Rhode Island, in all cases the census extending over an 8-day period. Counts upon two of these sections are given in detail in an accompanying table. Sundays being averaged, the 8-day period reveals the passage of the foregoing number of passenger cars and auto trucks at the points indicated in Table IV.

Worcester's Water-Works Pipe Trenching Machine

THE trenching machine bought by the Water Department of Worcester, Mass., in 1913, continues to give good service according to a paper by George W. Batchelder, water commissioner, read before the New England Water Works Association at the recent Holyoke convention. Besides being used in Worcester whenever practicable the machine has been rented for use in Hartford, Conn., and Quincy, Dartmouth and Auburn, Mass. The rentals thus far total $9,011, not including work now in progress at Auburn.

The machine is a model O Austin, and cost $6,650 in 1913. It has buckets 18, 24, 30 and 36 in. wide, with teeth projecting 3 in. on each side so that cuts 6 in. wider are made. By barring down material on both sides of the trench it is possible to make still wider cuts.

The total cost of repairs and replacements thus far has been $3,864 and the machine is apparently good for many more years' service, Mr. Batchelder states.

When rented out the city charges 7.55c. per cubic yard for the use of the machine.

	Length of Trench Dug, Ft.	Hours Work
Aug. 18—Very coarse gravel	410	8
Aug. 19—Loam, hardpan and gravel	250	5
Aug. 20—Hardpan, clay and sand	380	6½
Aug. 21—Sand and fine gravel	165	3
Aug. 23—Filled land, very rocky	384	6½
Aug. 24—Course gravel and sand	438	8
Aug. 25—Very rocky and wet	445	8½
Aug. 26—Fine gravel and hardpan	295	5½
Aug. 27—Gravel, clay and hardpan	472	8
Aug. 28—Filled land, rocky	180	3½

The above table is the record for the machine on trenches 2 ft. wide and 5 ft. deep at Auburn, Mass., in August, 1920.

Tree Planting Along California Highways

Tree planting along the highways in California, when the trees are of a kind approved by the state forester, is favored by the state highway commission and "where planted in accordance with the terms prescribed in the permits, will be maintained by the commission." When the commission receives inquiries about tree planting on the highways an application for a permit is supplied. When this application is filled out and returned a field investigation is made by the commission and if this results favorably a permit is issued. This permit prescribes the kind and size of trees, exact location, size of holes to be dug, replacing of earth and protection to be given, etc. The commission is planning the establishment of a nursery in conjunction with the California State Board of Forestry.

LETTERS TO THE EDITOR

Pro-Federation Arguments Attacked

Sir—A reading of the arguments that accompany the Am. Soc. C. E. ballot on the question of that society entering the new Federation of Engineering Societies brings out two facts:

First: There is no essential difference between the present proposal and that which was defeated by the society last spring.

Second: No positive and definite reasons for entering the Federation have been advanced in these arguments.

The plea for a "Yes" vote reveals the same vagueness as to aims and purposes that brought about the earlier defeat of the proposal, and its resubmission at this time falls little short of being an affront to the intelligence of the society.

The chief argument and in fact the basis of the entire plea for the Am. Soc. C. E. entering the Federation is that "everybody is doing it." As a matter of fact only five of the seventy-one societies represented at the Washington conference have as yet joined the new Federation.

On the other hand the American Chemical Society has refused to join and the American Association of Engineers has indicated that it will do likewise. It appears, then, that the unanimity of which so much is made by the advocates of the Federation is not in accordance with the facts.

In becoming a part of the Federation the Am. Soc. C. E. has nothing to gain and everything to lose. The inequitable organization of the Federation, its hazy purposes, its vague promises, and the utter lack of positive and convincing argument in its support, all demonstrate that it offers no benefits to the profession at large that can possibly justify the sacrifices the Am. Soc. C. E. is called on to make.

JOHN P. HOGAN,
CLIFFORD M. HOLLAND,
FREDERICK C. NOBLE.

New York City, Sept. 27.

Shrinkage of Earthwork

Sir—For the information of your readers who are interested in the shrinkage of earthwork the requirements contained in the specifications of the Mississippi River Commission for levee work is given as follows:

"*Shrinkage.*—The allowance for settling or shrinkage of embankment will be 15 per cent of the net fill for scraper work, 20 per cent for wagon work, and 25 per cent for other methods including dragline and other machine scrapers regardless of the material used; and no embankment that is not carried up as above specified will be paid for. The allowance for shrinkage or settling must be so disposed on top and slopes as to give the required width of crown, and to fill out the slopes so as to make plane surfaces from edge of crown to base of levee after shrinkage has practically ceased. The contents of the levee, computed to the established net grade only, will be paid for.

"*Settlement of Foundation.*—Should the contractor desire payment for restoration of yardage due to settling of foundation, he must apply to the contracting officer for instructions regarding the erection of structures for the determination thereof, in advance of any work on sections on which such payment may be made, and he must erect at his own expense, prior to any work thereon, such structures as may then be required by the contracting officer. Should these requirements be fulfilled the contractor will be paid for the restoration of yardage due to settlement of foundation at the same rate as the other embankment, but he will receive no compensation for such restoration unless the instructions of the contracting officer in regard to the determination of quantities shall have been fully complied with."

These requirements have been adopted after a great many years of experience in levee construction work and have

fairly met the general conditions under which these levees are built. The varying character of the soil or earth borrowed, the time required for and the weather conditions during construction, as well as the method of construction, are largely the controlling factors of shrinkage. The settlement of the foundation of levees and embankments is usual and in some cases very considerable and this should not be attributed to or confused with the shrinkage of the borrowed earth. In levee work measurements are usually made in embankment.

H. N. PHARR, Chief Engineer,
St. Francis Levee District.

Bridge Junction, Ark., Sept. 22.

Sir—The theories advanced by Charles Prelini in support of his contention that earthwork does not shrink, *Engineering News-Record*, Sept. 2, p. 475, are exceedingly ingenious but, unfortunately, they appear to be lacking in facts for a foundation.

In the writer's letter, published in your issue of Aug. 19, descriptive of work on the Yale Bowl, it was stated that "The embankment was mostly formed from material scraped from the enclosed area." Consequently, all of the material which was spilled from the scrapers fell where it was picked up by subsequent passages of that machine, and there was absolutely no waste in transportation.

The same is true as regards material lost by erosion. In ordinary storms the water was readily absorbed by the embankment and it was only in rainfalls of exceptional severity that any of the sand was displaced. The small amount of material which was eroded was either washed down into the cut, from where it was subsequently replaced, or else caught and retained on the embankment by the concrete wall which surrounds the Bowl at the foot of the outer slope.

A theory to account for the difference in volume of excavation and embankment, in the case of the Yale Bowl, must be other than that due to loss of material either in transportation or by erosion.

CHARLES A. FERRY,
Engineer, Yale Bowl.

New Haven, Conn., Sept. 14.

Venturi Tube Suggested for Holding Vacuum in Siphon

Sir—The writer was much interested in a short article in the Sept. 16 issue of *Engineering News-Record*, p. 541, in which the use of a true siphon on a main supply pipe was described. The conditions in this case were comparable to those encountered by the writer about fourteen years ago where it was desired that a 16-in. riveted steel pipe line should be placed temporarily in service pending the completion of a pumping plant. A long crest with a maximum height of about 6 ft. above hydraulic gradient began a short distance from the intake of the pipe line and it occurred to the writer that it would be possible to siphon the water across this crest. With this end in view, a connection at the highest point in the line was provided, above which was placed an air chamber about 2 ft. diameter and 3 ft. high. A gage glass was placed on the side of the air chamber, near the bottom, so that the attendant could see when the surface of the water in the chamber approached the level of the top of the 16-inch. pipe.

When the chamber was pretty well filled with air, work was started with a hand-operated air pump (as I remember it, one of the same type generally used in garages years ago before the advent of the power-operated tire-pump) and by this rather laborious means the air would be cleared from the chamber. Of course, so long as the level of the water in the chamber was above the connection with the line the flow would be complete.

The only trouble we had in this case was the difficulty in preventing leakage of air into the riveted pipe line, as it didn't take much to keep the air pump at work pretty constantly. With this exception, however, the scheme was a success.

It has occurred to the writer, however, that in such cases as this, or the one described by Mr. Braun and Mr. Sherman, it would be easily possible to keep the air chamber clear of air by installing a venturi tube just below the hydraulic gradient beyond the crest (toward the discharge end of the line) and connecting the throat with the air chamber with the small tube. In the case described in the articles referred to a venturi tube for this 6-in. pipe, having a 2-in. throat, should, with a velocity through the full section of the pipe of 2 ft. per second, give a differential head more than sufficient to keep the chamber clear. This would mean a rate of discharge of but 175 gal. per minute. As a matter of fact, if the crest is but 3 ft. above the hydraulic gradient, it would be found that the required degree of vacuum would be about balanced by the throat velocity due to a discharge of about 90 gal. per minute.

While the use of the siphon has been very generally avoided in the past for very obvious reasons, in such cases as those referred to above the writer believes that it would be easily possible to adapt the venturi tube to the work of holding the required vacuum and offers the suggestion for what it may be worth.

GEORGE SYDNEY BINCKLEY.

New York City, Sept. 27.

Engineers in Public Service Would Gain by Uniform Classification

Sir—Engineering Council's proposed classification and compensation of engineers in federal, state, county and railway service is to my mind one of the most important movements recently put forth to bring about a higher state of efficiency in the engineering service of our public and semi-public institutions. I believe the same inducements should be offered to men entering the above branches of engineering as are offered to those entering private fields; namely, chances for continuous promotion based on the individual's ability and efforts.

At the present time the engineering field in public service is extremely limited. The engineer of ambition in a small town has very little hope after he reaches a high position there of advancing to a more responsible position in the larger town or city, for the reason that local civil service rules form a barrier through which he cannot pass without starting in the lower grades and working up in the particular town or community in which these rules are operating.

The universal adoption of a standard classification and compensation of engineers, such as proposed by Engineering Council, would be the first step forward by which the present complicated and inconsistent civil service grading might be abolished and in its stead a uniform system of grading and compensation established throughout the country. This would be a most important advance step, as it would pave the way for making engineering in the public service one large field where the engineer might start in Oshkosh as a "junior aid" or "junior assistant engineer" and work up through the various grades, ultimately reaching the larger and more important organizations in our larger municipalities or in the federal, state or county service.

It requires no unusual range of vision to picture a crying need for competent engineers who have been trained in the public service to properly and efficiently design, construct and maintain public works and to manage many of our cities. Every effort should be made now to lay a substantial foundation on which this important service to the community may be built. Every engineer and every technical society, whether it be an association, an organization or a union, should agree upon and get behind some such classification and compensation of engineers, as proposed by Engineering Council. When we have agreed upon this—and I think we have pretty nearly agreed now, at least in so far as the proposed classification is concerned—let publicity work begin. Let us go at this publicity work in an engineering-like manner, organizing in every town, city and state where engineers are employed on public work. Let us get to the newspapers, magazines, etc., and through them show the public in their own language the community value they will derive from making federal, state, county,

municipal and possibly railway engineering service one big field within which the best engineers will gravitate to the top if they are offered the same incentive as now exists in private fields, in addition to that very strong incentive for public service.

This is not an impossible dream; it is a worthy object in which the entire engineering profession should be interested, interested enough to make the public interested. It's part of our job to educate the public to the benefits that may be derived from the adoption of improved methods in handling public affairs. It's time, anyhow, that we began to talk non-technical language so some one besides engineers will clearly understand the importance of our economic and social functions.

Let's begin now! HARRY W. LEVY,
Department of Public Works, Borough of Manhattan.
New York City, Sept. 7.

Smooth Approaches and Floors Will Reduce Highway Bridge Impact

Sir—I read the editorial in your Sept. 7 issue regarding the necessity for a study of impact and its benefit on highway bridges. I think all highway engineers thoroughly appreciate the importance of this subject, for at the present time there is no information which would allow us to arrive at the result of the amount of stress generated by a moving load striking a bridge. There are so many factors that enter into this proposition that it seems to me it would be an extremely difficult and complicated matter to secure any formulas which would apply.

For practical purposes, it is my opinion that highway bridge engineers at the present time should take every possible precaution to make the approaches to bridges smooth, so that the impact from a load striking a bridge will be at the minimum. They should also see that the floors of bridges are laid properly and maintained in such shape that they may not be rough and uneven. In this way we can immediately preserve to the public the service of certain bridges which otherwise might be wrecked due to impact stresses.

It is my opinion, based on some practical experiences, that there are bridges which might fail if the floors are left uneven and rough, but which might easily carry the loads to which they are subjected if the floors are properly maintained, so that during the time that we are making investigations of theories, we should see that what we have is properly taken care of. C. J. BENNETT,
State Highway Commissioner.
Hartford, Conn., Sept. 14.

What Is Art?

Sir—Referring to the letter of John C. Trautwine, Jr., in the issue of Sept. 23, p. 619, under your heading "What Is Art?" I beg to subscribe to his statements and to the quotation from President Charles MacDonald's address at the Denver convention of the American Society of Civil Engineers in 1908.

When writing my thesis over 53 years ago, I selected the subject: "Aesthetics Applied to Bridge Design." A proposition was set forth therein, based largely on Greek and Roman art, that the lines and forms which appeared most beautiful to us were those which would most faithfully represent the internal stresses caused by the external forces acting upon the structure, with the least amount of material and arranged in the most economical way.

The internal forces result from the resistance of the material and the external forces from the loads upon the structure, both static and dynamic and from the wind, rain, and extremes of temperature. The measurements and the proportioning of all these forces are purely scientific, but form the artistic foundation.

To restate, the greatest beauty of form in a structure and its parts is secured by giving the required resistance to every attacking force with the least amount of material, and in the simplest and most direct way.

As a monstrous example of the combination of architecture and engineering in bridge construction, visitors to the Philadelphia World's Fair in 1876 may remember the double-decked bridge across the Schuylkill River of 400 ft. span. It is a double-intersection Pratt truss, which was then artistically (?) decorated on both sides by a sham false plating of cast iron ⅛ to ¼ in. thick, intended to resemble a roman arcade crossing the river. The columns enclosed the posts and rested on pedestals joined by a balustrade all resting upon the lower chord. The columns were joined at the top by semi-circular arches just below the upper chord. The arcade rested on air and the diagonal tie-rods were in full view between the columns!

Fortunately, both for art and engineering, the weather soon rusted the thin iron sham and boys finished the job by practicing football against the corroding balustrade. Since removing the iron sham work the whole truss creditably and artistically shows what it is, without architectural embellishment, which, I feel sure, the Romans would never have endorsed and Americans will never repeat.

RUDOLPH HERING,
Consulting Engineer.
New York, Sept. 25, 1920.

Effect of Illumination on Two-Photograph Method of Detecting Motion

Sir—In your issue of Aug. 26, 1920, page 409, there is an article under the title "Changes in Configuration Revealed by Superposing Photographs." It is mentioned in the article that, to make effective this method of detecting changes in configuration by the superposition of positive and negative plates, the photographs must be taken from the same position, and the same camera and kind of plates should be used.

Another very important element to make this method successful would be that the exposures be taken under about the same direction of illumination. The method depends essentially on the comparison of lights and shadows, and it is apparent that the lights and shadows on a panorama taken with the sun in the east would be very different from the same view taken with the sun in the south or with the sun in the west. The effect of direction of illumination is very marked in the illumination of ornamentation on a building or other artistic objects due to the movement of lights and shadows under the different directions of illumination. It would appear to have an important bearing on the method outlined in the article. S. H. GRAUTEN,
Electrical Engineer, Kansas City Railways Co.
Kansas City, Mo., Sept. 2.

Dry Out Sweating Vault

Sir—The letter in the *Engineering News-Record*, Sept. 9, p. 521, in reference to the sweating of the concrete vault, is very interesting to us for the reason that we have had somewhat similar experiences in the past.

We do not know all of the conditions surrounding the construction of the vault but we are inclined to believe that a large part of the trouble is due to the fact that it has never been dried out, and we feel that the sweating would be materially reduced if the vault had once been dried out by the application of heat.

We have also conferred with an architect who has had a very extensive experience in the construction of banks and he advises us that he has never succeeded in constructing a vault below grade which did not sweat. This architect states that he has constructed a great many vaults which were waterproofed but on account of the difference in temperature between the walls and the interiors of the buildings the vaults continued to sweat. His practice, where a dry vault is needed, is to install a small electrical heater and an expeller fan; then every few days the heater is turned on to raise the temperature and the fan is started and the vault dried out. As soon as any noticeable amount of moisture collects again the process is repeated.

CHARLES F. DINGMAN,
Engineer, Flynt Building Construction Co.
Palmer, Mass., Sept. 16.

NEWS OF THE WEEK

New York, September 30, 1920

Local Financing in South America Noted by S. T. Henry

Before the war South America depended largely on European capital to finance engineering work, but during the time that funds from foreign sources have not been available the plan of raising money locally has succeeded in many instances. This departure from precedent was observed by S. T. Henry, vice-president of the Allied Machinery Co. of America, who has just returned to this country after a six months' trip through South America and who, last week, described his experiences at a meeting in New York City of the members of the Allied organization.

If local financing can be further stimulated, as now appears probable, Mr. Henry believes that South America will afford many opportunities for contrators with modern construction equipment. The increased cost of labor has emphasized the value of mechanical plant, especially in view of the fact that the Latin American worker is combining a decreased efficiency with insistence on an 8-hour day. Under such conditions local contractors are realizing that the use of modern, labor-saving equipment is essential to the making of profits.

Five-Inch Pavement, Reinforced, Now California Minimum

At a recent meeting of the California Highway Commission it was voted that "hereafter, because of the rapidly increasing volume and intensity of the traffic over the state highways, no concrete base shall be laid on any state highway a lesser thickness than 5 in. and all concrete bases are to be reinforced with steel."

The instructions as to the method of placing the steel and its dimensions as explained by A. B. Fletcher, state highway engineer, are as follows:

"On all future day labor authorizations and contracts for the construction of concrete bases and for concrete shoulders the outer edges shall be reinforced by ½-in. square deformed steel bars placed longitudinally along the edges, centered in the depth of the slab and placed 2 in. from the outer edges. Such longitudinal bars shall be lapped 12 in. for bond. At intervals not greater than 30 ft. the bars shall be butted and not lapped. Transverse reinforcement shall consist of ⅜-in. square deformed steel bars placed transversely on 18-in. centers, such transverse bars being hooked over the longitudinal bars and securely wired thereto."

No Increased Expense from Amendments, Says President Davis

Arthur P. Davis, president of the American Society of Civil Engineers, has sent to *Engineering News-Record* the following reply to the communication by Willard T. Chevalier, published as a letter to the editor in last week's issue:

"Replying to the communication of W. T. Chevalier in your issue of Sept. 23, p. 617, the proposed amendments to the constitution would not commit the society to additional expense because the expenses of the Nominating Committee are paid now. If the new committee is larger this is more than offset by the reduction in the Board of Direction, which meets eight times a year to once for the Nominating Committee. The amendment provides for payment of expenses for only one delegate from each section, and that only once a year. This might slightly increase expenses if amendment F is defeated but not otherwise. I am informed that of the five members of the Northwestern Section who signed the "Appeal" four will vote for the amendments criticized after having studied them further. A similar proportion will probably obtain in other regions."

State Highway Officials to Meet

The American Association of State Highway Officials will meet in Washington Dec. 6 for a four-day convention. The decision to call this meeting was reached at a session of the executive committee of the association which met in Washington Sept. 21.

The executive committee conferred at length with Thomas H. MacDonald, chief of the U. S. Bureau of Public Roads. They also had a round table discussion with the representatives of a number of manufacturers of motor trucks. The truck manufacturers advanced the suggestion that an educational campaign should be conducted among the users of trucks as to just what they should do to avoid damaging highways. If owners of trucks would do all in their power to reduce to a minimum the chance that their truck may damage a roadway, it is believed that they will be spared many burdensome regulations which are believed certain to come unless prompt steps are taken to reduce the amount of damage being caused by trucks. During the course of the conference a visit was paid to the experimental station of the Bureau of Public Roads at Arlington.

Urge Am. Soc. C. E. Members To Vote "Yes"

Proposed Amendments to Constitution of Society Endorsed in Printed Appeal Signed by 362 Men

Arguments in favor of the proposed amendments to the constitution of the American Society of Civil Engineers, which are before the members in the form of a referendum to be canvassed Oct. 6, are being distributed as a printed 4-page pamphlet addressed to corporate members and carrying 362 signatures. The communication follows:

The undersigned who are working for a really progressive policy in the society, inspired with a desire to add to the traditions and advance the ideals of the society, have followed the work of its Committee on Development with the deepest interest and are looking forward to the adoption of the proposed amendments, which are based upon its recommendations, as a proper means for advancing the professional standing and increasing the prestige of the society.

We appeal to the pride and deliberate judgment of the corporate members who desire a progressive policy that is in keeping with the time to weigh the facts carefully, believing it is their wish overwhelmingly expressed by the questionnaire ballot, canvassed April 14, 1920, that the society shall become "an active national force in economic, industrial and civic affairs"; and that the directors in each geographical district shall be nominated and elected by vote of the corporate members resident therein.

With the object of clarifying the present situation the following facts are set forth:

The Board of Direction submitted to the society amendments "A," "B," "C" and the substance of amendment "G" which had been reported by its special committee with the following comment:

"In preparing the proposals for amendments to the constitution submitted herewith, the special committee appointed by the Board of Direction has thought it best that the changes in the constitution of the society be limited at this time to those necessary to make effective the recommendations of the Committee on Development specifically approved by vote of April 14, 1920, upon the questionnaire."

Amendment A—It is urged as a reason for the defeat of this amendment that the society by undertaking "to co-operate in important economic, industrial, and civic movements," would cease to be a purely technical society and would thus be liable to taxation. A recent legal opinion, which the Board of Direction can confirm, shows that this statement is not true.

Amendment B—This amendment is verbatim as submitted by the Board of Direction. This proposal to assign all members of the society to local sections was approved by a 2½ to 1 vote of the society, April 14, 1920.

Amendment F—This amendment remedies a great injustice. The constitution provides that there shall be included in District No. 1 the corporate members resident in the territory within fifty miles of the Post Office in New York City, or according to the latest year book, 1,248 members. Notwithstanding this fact there are included in District No. 1, 933 corporate members living outside of North America, making the total membership included in District No. 1, 2,181. District No. 1 now has a minimum representation on the Board of Direction of 30 per cent of the board's membership for 15 per cent of the corporate membership of the society resident in the district. In other districts the proportionate representation is much less. For example, in District No. 5 for over 9½ per cent of the corporate membership there is a minimum representation on the Board of 3⅓ percent. The proposed amendment reduces the minimum representation of District No. 1 on the board to 16⅔ per cent for 15 per cent of the corporate membership.

It frequently happens that District No. 1 has a president, two vice-presidents and two or more past-presidents and then its representation exceeds 40 per cent of the membership of the board for 15 per cent of the corporate membership. The New York District, because of this excessive representation, has been able to control the policies of the society. It is contended that this amendment is in conflict with the constitution, which provides that members of the society shall be elected by at least 25 members of the board, whereas the proposed amendment provides only 24 members. The proposed amendment provides that all officers of the society at the time of its adoption shall serve to the end of the terms of office for which they were elected. For the year 1921 there will, therefore, be 26 members on the board; 25 in 1922; and 24 members in 1923. The proposed amendment provides the requisite number of directors for the election of members for more than two years, which gives ample time to further revise the present constitution.

Amendment G—Article VII, Section 2, differs in details as to the form of procedure as submitted by the Board of Direction. The election of the directors by the corporate membership of the geographical districts which they are to represent was recommended by the Conference of Presidents of Local Associations in 1915, was approved by a more than 4 to 1 vote of the society canvassed by the board in 1916, and by a 5½ to 1 vote of the society canvassed April 14, 1920.

Section 3 of Article VII and Section 8 of Article VIII, providing for an Annual Conference of the representatives of local Sections as recommended by the Committee on Development, were approved by a 2½ to 1 vote of the society April 14, 1920.

Sections 4, 5 and 6 are substantially the same as those in the present Constitution.

Amendments "A," "B" and "G" are based upon the recommendations of the Committee on Development which were approved by a 3 to 1 vote of the 1920 Annual Meeting of the society, by a minimum 2½ to 1 ballot canvassed April 14, 1920, and by the 1920 Annual Convention by a vote of nearly 6 to 1. Those who have controlled the government of the society for the past twenty-five years now seek to defeat the expressed wishes of the society through unfavorable sentiment for the proposed amendments backed by arguments unsupported by facts and at variance with the spirit of these amendments.

"An Appeal for a Progressive as Against a Radical Policy" is in reality an entreaty to the membership to defeat the amendments and thus allow the present objectionable status to continue. It is stated: "These amendments are not those formulated by the committee of the Board of Direction appointed for that purpose. They are almost entirely the product of the radical faction of the society." On the contrary, one-half of the amendments the members are asked to vote against were prepared by a special committee of the Board of Direction. These amendments instead of establishing a political machine embody changes which are intended to democratize the society, and decentralize authority. The recommendations of the Committee on Development were accompanied by a budget, the result of careful study made after consultation with the secretary of the society; this budget showed that an increase of $5 per annum in the dues of non-resident corporate members would provide sufficient funds to cover the increased expense resulting from putting its recommendations into effect.

It has been stated that it might cost $16,000 a year for the expenses of delegates to the annual conference of representatives of local sections, on the basis of 80 sections. This statement does not seem warranted in view of the following facts:

1. The society is now liable for the expenses of 19 members of the Committee on Nomination.

2. There are 25 local sections in the society.

3. The budget, accompanying the report of the Committee on Development, contained an estimate of $3,520 for the expenses of the delegates of 40 sections; from this amount was deducted the expenses of the members of the Committee on Nominations making a net increased expense of $1,848 for an Annual Conference of the delegates of 40 sections.

4. The American Society of Mechanical Engineers, with over 12,000 members, had 38 sections represented at its last annual meeting and the cost was about $1,500; $3,500 is the estimated expense for the next annual meeting. Present allowance 10 cents per mile and $4 per day at the meeting.

5. The American Institute of Electrical Engineers, with over 11,000 members, has 36 sections and the expense of the 30 delegates to the annual convention, June 29 to July 2, 1920, was $2,500. Allowance railroad and Pullman fares and meals en route.

The proposed amendments provide a definite recognition for local sections—which they do not now enjoy. The details of administration are left to the Board of Direction, where they customarily belong.

The special committee authorized by the Portland Convention was not created for the purpose of making a general revision of the constitution, or to employ counsel to pass upon the legality of the amendments, but merely to report upon certain specific amendments considered by the convention and which are not involved in the present ballot.

The progressive development of the society must be accomplished step by step. The vote on the questionnaire April 14, 1920, emphatically indicated that the corporate membership of the society approved the progressive policy recommended by the Committee on Development. The proposed amendments are the next step in carrying out this progressive policy. Do not be induced by any appeal to vote against these amendments. Their defeat will indefinitely delay this progressive movement.

Every corporate member of the society should read President Arthur P. Davis' letter in Sept. 16, 1920 issue, *Engineering News-Record* (pp. 570 and 573).

If you have already voted you may withdraw your vote by dating, and endorsing over your own signature (handwritten not typewritten or printed) on the outer envelope in which you mail your latest ballot "Destroy previous ballot"; the ballot should be mailed not later than Sept. 30, 1920.

On behalf of real progress in the society the undersigned appeal to you to vote *Yes* for the proposed amendments:

E. L. Adams, Louis J. Affelder, J. C. Akers, T. D. Allin, J. C. Allison, George G. Anderson, J. H. M. Andrews, H. C. Ash, Richard I. D. Ashbridge, Lionel Ayres.

Percival S. Baker, H. G. Balcom, Walter F. Ballinger, A. F. Barnard, E. C. Barnard, W. K. Barnard, Francis Bates, George L. Bean, F. Phillips Best, Thomas B. Blackburn, Louis P. Blum, C. K. Bowen, E. R. Bowen, S. W. Bowen, A. W. Bowie, Baxter L. Brown, Norman F. Brown, George Bryan, Jr., Morton Burden, C. A. Burnette, George W. Burpee, W. J. Burton, A. D. Butler, Gordon Butler.

H. F. Cameron, Charles C. Campbell, John F. Campbell, Horace T. Campion, C. M. Canady, John Carson, A. A. Casani, F. D. Cefalu, E. C. Chamberlin, Ralph H. Chambers, C. T. Chenery, J. N. Chester, William H. Chorlton, C. D. Christie, George L. Christie, H. L. Christie, F. B. Church, Leland Clapper, Edwin Clark, W. A. Clark, Francis A. Cokefair, E. H. Coleman, R. S. Colnon, William H. Connell, John S. Conway, William H. Crawford, E. W. Crellin, C. M. Daily, A. M. Danzilli, J. H. Darling, A. L. Davis, C. E. Davis, D. E. Davis, Edwin F. Dawson, E. F. Delery, F. E. Dempsey, C. M. Denise, H. W. Dennis, F. G. Dessery, Richard G. Develin, James L.

de Vou, E. C. Dicke, O. H. Dickerson, Clark Dillenbeck, E. C. Dilworth, H. J. Doolittle, W. S. Doyle, E. H. Dresser, F. L. Dudley, Arthur J. Dyer.

William Easby, Jr., Samuel Eckles, J. H. Elder.

Edward B. Fay, F. H. Finck, Wager Fisher, G. C. Fitzgerald, Edwin J. Fitzmaurice, J. C. Fowler, Benjamin Franklin, Francis E. Freeland, Charles Frommer, George S. Frost, William C. Furber.

B. J. Garnett, J. T. Garrett, R. P. Garrett, O. H. Gentner, Jr., Elmer A. Gibbs, W. Herbert Gibson, Frank Gillelen, Irving D. Goodwin, A. E. Goodwin, R. R. Gould, R. R. Graham, William H. Gravell, F. W. Green, A. P. Greensfelder, William B. Gregory, Charles F. Gross, William G. Grove, N. C. Grover.

Albert B. Hager, John P. Hallahan, Milo C. Halsey, T. R. Halsey, John W. Hamilton, Charles Hansel, Andrew B. Hargis, C. H. Harlan, George R. Harlow, E. L. Harman, Stephen Harris, O. F. Harting, F. A. Hastings, Harry Hawgood, William E. Hawley, F. Henderson, V. K. Hendricks, Clemens Herschel, Harry A. Hickman, Louis C. Hill, Arthur Hirst, L. R. Hjorth, W. W. Horner, A. H. Horton, F. E. House, John J. L. Houston, Otis E. Hovey, R. H. Howard, John C. Hoyt, William H. Hoyt, Richard L. Humphrey, A. M. Hunt.

J. Granberry Jackson, Nathan B. Jacobs, Reginald H. James, Edwin S. Jarrett, A. N. Johnson, Alfred Jones, H. L. Jones, S. A. Jubb.

Alfred L. Kehoe, E. W. Kelly, Richard Khuen, Jr., G. W. Kinne, B. H. Klyce, Morris Knowles, William J. Kupec.

John F. Laboon, C. G. Emil Larsson, Harrison W. Latta, Ralph J. Lawrence, J. A. Lewrie, A. L. Lee, Marshall O. Leighton, S. H. Leister, Horace G. Leng, J. B. Leeper, W. I. Lex, A. R. Lindsay, C. E. Long, F. W. Lyon.

I. W. McConnell, H. E. McCool, Hunter McDonald, Morton Macartney, Charles MacDonald, John MacMillan, L. T. Maenner, R. G. Manning, Charles W. Martin, John Meigs, O. C. Merrill, William F. Miller, J. H. Miner, W. S. Mitchell, Frederic A. Molitor, Carl deMoll, Charles E. Mollard, Burdett Moody, F. C. Moore, Walter Moore, Jr., S. B. Morris, John F. Murray, William S. Nichols, C. W. Ogden, Ole K. Olson, F. H. Olmstead, William P. Parker, W. B. Patton, Walter Pearl, Emile G. Perrot, Arthur I. Perry, J. L. Pickles, Henry S. Prichard, B. B. Priest, Charles F. Puff, Jr., Marshall R. Pugh, C. D. Purdon, Clyde B. Pyle.

Earl Querback, Henry H. Quimby.

J. C. Ralston, R. J. Reed, P. J. Reich, John M. Rice, H. Ridgway, John E. Rockhold, W. E. Rolfe, Alstin O. Rose, George F. Rowell, J. W. Rowland, E. J. Ruff, S. Bent Russell.

C. W. S. Sammelman, Maurice R. Scharff, Nathan Schein, G. M. Schofield, H. H. Sears, H. J. Sharp, A. M. Shaw, William J. Shea, E. G. Sheibley, H. T. Shelley, E. H. Shipman, L. H. Shoemaker, Joseph G. Shryock, Joseph W. Silliman, J. W. Skelly, Joseph M. Slater, S. A. Sloan, C. E. Smith, Henry C. Smith, Pemberton Smith, Stanley Smith, A. L. Sonderegger, Ellis C. Soper, W. H. Spear, T. Nelson Spencer, N. S. Sprague, Herman Stabler, J. R. Stack, Norman L. Stamm, F. Charles Starr, Herbert H. Starr, Edward B. Stearns, F. LeRoy Stearns, Charles H. Stevens, John D. Stevenson, H. C. Stowe, H. Struckmann, J. T. Stuart, C. R. Sumner, S. M. Swaab.

E. G. Taber, C. M. Talbert, G. A. Taylor, G. Lewis Taylor, E. B. Temple, J. Hermon Terry, Franklin Thomas, C. J. Tilden, F. E. Trask, A. J. Turner, Daniel L. Turner.

J. H. Van Wagenen, Paul Voorhees, C. B. Voynow, R. E. Wachter, J. E. Wadsworth, Joseph C. Wagner, Samuel T. Wagner, O. A. Wah, J. J. Walker, J. S. Walker, Edward E. Wall, Albert K. Warren, C. L. Warwick, L. C. Wason, George S. Webster, Maurice A. Webster, William A. Weldin, R. C. White, W. M. White, W. E. Whittier, E. H. Wilcox, Frank L. Wilcox, T. J. Wilkerson, Marshall Williams, W. H. Williams, C. B. Wilson, D. R. Wilson, Percy H. Wilson, Lef. Winship, Francis P. Witmer, F. C. Woermann, J. W. Woermann, S. L. Wonson, W. H. Woodbury, Wilkie Woodward, Robert B. Woodworth, J. C. Wright, S. H. Wright, Alfred M. Wyman.

Walter G. Zimmermann.

A supplementary list of names received too late for printing in the foregoing pamphlet has been sent to *Engineering News-Record*. It follows:

A. L. Ackhart, Willard Beahan, John Boldt, C. P. Bower, J. H. Brillhart, Wendell P. Brown.

William J. Carter, H. W. Dimer, Charles Y. Dixon.

J. G. Esch, H. H. Esselstyn, P. P. Evans, Herman Fougner, Harry Fuller.

F. L. Gorman, L. M. Gram, Bernard L. Green.

R. L. Harding, R. Husselman, Edward Hutchins.

Theodore A. Leisen, A. G. Levy, John E. A. Linders.

Philip O. Macqueen, W. M. Mitchell, J. E. A. Moore, Edgar B. Morris, David W. Morrow.

K. H. Osborn, R. B. Perrine.

Frederick D. Richards, James Ritchie, A. V. Ruggles.

James C. F. Shafer, Huntington Smith, F. H. Stephenson, D. Y. Swaty.

George H. Tinker, F. J. Van Hook.

E. M. Walker, W. J. Watson, D. R. Wells.

Am. Soc. C. E. Amendments Supported

In addition to the pamphlet urging the adoption of the constitutional amendments of the American Society of Civil Engineers, published in another column, the following communication has been sent in printed form to the corporate membership:

Amendments A, B, C and G to the constitution of our society, now before the membership, are the first step in carrying out those recommendations of the Committee on Development which were unanimously adopted after two years of study and debate by its members and which have been approved by the vote on the Questionnaire canvassed April 14, 1920.

Amendment A is covered by Question A1, which was approved by a vote of 2,889 to 910.

Amendment B is covered by Question B2, which was approved by a vote of 2,854 to 831.

Amendment C is covered by Question B6, which was approved by a vote of 2,583 to 1,149.

Amendment G is covered by Questions B3 and B5, which were approved by votes of 3,176 to 567 and 2,613 to 1,091, respectively.

Amendment F reduces the number of directors from 18 to 15 by reducing the representation of District No. 1 from 6 to 3; removes the secretary from membership on the Board of Direction; reduces the number of past presidents on the board from 5 to 3; and eliminates the requirement that at least one vice-president shall be a resident of District No. 1.

It brings squarely before the membership the question of whether in the future it shall be possible that District No. 1 with a resident corporate membership of 1,248 shall have as heretofore 12 votes out of 30 on the Board of Direction, giving it practically 1 for each 100 members as against 18 votes for 6,965 non-resident members, or less than 1 for each 380 thereof.

The question may be stated thus: Shall a small minority in New York City have, as in the past, virtual control of the affairs of a society having 85 per cent of its membership outside of that city, or shall the representation be so readjusted that the New York membership of the Board will be at least four, or one for every 316 members against a possible maximum of twenty for the non-resident members or one for each 364 on the basis of present numbers, and the governing body of the society become thereby truly national in its makeup?

Prof. Ira O. Baker, Mortimer G. Barnes, Prof. John H. Bateman, Prof. Arthur H. Blanchard, J. H. Brillhart, H. R. Buck, A. D. Butler.

John F. Coleman, Edward H. Connor, Mortimer E. Cooley.

William L. Darling.

John Ericson.

William M. Gardner, John E. Greiner.

H. L. Hachl, John L. Hall, Charles Hansel, E. E. Haskell, John B. Hawley, W. H. Hoyt, Maj. W. C. Hoad, Clarence W. Hubbell.

Prof. Clarence T. Johnston.

Richard Khuen, Jr.

Maj. Theodore A. Leisen.

Hunter McDonald, George C. Mason David A. Molitor.

Frank C. Osborn.

Arthur Pew, George S. Pierson.

J. C. Ralston, Prof. Henry E. Riggs.

E. C. Shankland, F. H. Stephenson.

E. B. Thomas, Prof. C. J. Tilden.

E. M. Walker, E. E. Wall, Dalton R. Wells, Gardner S. Williams, George M. Wisner.

New Signatures to Supplementary Appeal Total 33

The following 33 additional signatures to the supplementary "Appeal" for administrative reforms in the American Society of Civil Engineers, as published in *Engineering News-Record*, Sept. 23, p. 617, have been received, this making a total of 115 names. This journal is informed that some of the names on the original "Appeal" are not on the supplementary circular because the subject is now under discussion in the local section to which the signers belong. Other signers, it is said, are not wholly in sympathy with the reforms advocated. The claim is made that many more have not responded because of the time limit that was set for the receipt of replies.

S. Barfoed, E. I. Clawiter, M. C. Couchot, Charles Derleth, Jr., E. C. Hutchinson, F. R. Muhs, M. M. O'Shaughnessy, S. B. Smith, J. J. Walsh, C. F. Loweth, W. L. Breckenridge, D. G. Brumley, C. B. Burdick, C. F. W. Felt, A. W. Newton, R. I. Randolph.

L. K. Sherman, C. L. Strobel, D. H. Maury, Elbert Reichmann, Edwin C. Finley, E. M. Hoopes, F. W. Cappelen, J. D. Wardle, R. T. Hartman, A. R. Swem, H. R. Green, J. R. Worcester, C. T. Main, F. M. Gunby F. H. Fay, C. M. Spofford, C. R. Gow.

Ventilating Shaft Contract Awarded to Bryson

The first construction contract on the Hudson River vehicular tunnel, involving two ventilating shafts at the New York City end of the tubes, as described in this journal Sept. 23, p. 623, has been awarded to Thomas B. Bryson, whose bid of $650,802 was the lowest of the five opened Sept. 21.

Barber Asphalt Plant Burns

Fire, the origin of which as yet is undetermined, destroyed the Barber Asphalt Paving Company's plant at Maurer, near Perth Amboy, N. J., Sept. 26, causing damage which a preliminary estimate places in excess of $2,000,000. The fire spread over an area of six acres and, due to the dense smoke, caused from the explosions of oil tank cars, with difficulty was kept from spreading to adjoining manufacturing establishments, firemen being aided in fighting flames with searchlights.

Program Issued for Municipal Improvements Convention

The program has been issued for the 26th annual convention of the American Society for Municipal Improvements, to be held at the Planters Hotel, St. Louis, Mo., Oct. 12-15. The first day will be devoted to meetings of committees on specifications, to the reports of the officers of the society and to committee reports of a technical nature, such as water-works, street cleaning, etc. There is also scheduled for Oct. 12, a round-table luncheon discussion of the society's activities. The plan is for groups interested in certain subjects to gather about assigned tables, each of which will be in charge of a chairman.

A heavy schedule of technical papers and committee reports is planned for the remaining days of the convention. On the night of Oct. 14, the "Engineer Players," a group of members of the Engineers' Club of St. Louis, will present the morality play "Every Engineer." Other entertainment features include a golf tournament on Oct. 11, a reception and dance on Oct. 12, and an automobile sight-seeing trip.

Testing Material Standards Adopted

By letter ballot canvassed Sept. 1, 1920, the 23 tentative standards of the American Society for Testing Materials, accepted at the annual meeting in June, were adopted as standard. In addition the letter ballot was favorable to the revision in the Standard Specifications and Tests for Portland Cement providing for a 78 per cent fineness through a 200-mesh sieve. This becomes effective Jan. 1, 1921. The maximum vote cast for any standard was 158 and the maximum against 12, although the membership of the society is now well over 2800.

Belle Isle Bridge Plans Adopted

Detailed plans for the Belle Isle bridge, prepared by Esselstyn, Murphy & Hanford, engineers, have been adopted by the Detroit city council and it is intended to advertise for bids to be opened about Oct. 22 or Nov. 1. The engineers believe that if bids are opened before Nov. 1 it will be possible for contractors to have the work on the piers well under way before cold weather, and if the winter is an ordinary one it will be possible to continue work on the piers and foundations all winter. By the end of 1921 the piers should be ready for the steelwork and the bridge could then be completed by 1923, or even sooner if the conditions governing the supply of building materials should improve.

The bridge will be approximately 2,200 ft. long, comprising 20 spans 74 ft. to 135 long. The total width will be 88 ft., with 59 ft. roadway (asphalt pavement) for six lines of traffic and two 12-ft. sidewalks. No drawspan is contemplated, but the structure will afford an underclearance of 30 ft., sufficient for fire boats and pleasure vessels.

The detail plans have been made from the general designs submitted by a bridge commission made up of M. E. Cooley, University of Michigan, chairman; Prof. Emil Lorch, Prof. H. E. Riggs, Francis McMath, and William Kales. Prof. L. M. Gram was engineer for the commission. The cost of the structure is estimated at $2,000,000.

Board of Engineers to Report on Delaware River Bridge

A board of three engineers to make a traffic, location and design survey and report on a highway bridge over the Delaware River at Philadelphia was appointed on Sept. 23 by the interstate bridge and tunnel commissions of New Jersey and Pennsylvania in joint session. The board is composed of Ralph Modjeski, consulting engineer, New York; George S. Webster, chief of the Bureau of Surveys, Philadelphia; and L. A. Ball, consulting engineer, New York. Mr. Ball, a resident of East Orange, N. J., was selected by the New Jersey authorities to represent that state. Mr. Modjeski is named chairman of the board, at a fee of $20,000, while the other two members are to receive $15,000. It is not required that the members give their full time to the work.

For the survey and report an appropriation of $100,000 has been made by the commissions, this sum including the fees of the board members. In prior meetings the commissions received offers from various engineers in private practice to make such a survey and report thereon for approximately the amount stated, as reported in our issue of Aug. 19, 1920, p. 383. It is expected that the report will be completed within six or eight months. Three sites are to be investigated, one located south of Market St., the other two north of Market. Borings to disclose the foundation material, studies of property values and an estimate of the cost of the bridge are to be included in the report.

Declare Molasses Tank Safe; Dynamite Suspected

(Special Correspondence)

Expert witnesses testifying for the defense in the trial of damage claims against the United States Industrial Alcohol Co. for the deaths and injuries from the collapse of a large molasses tank in Boston on Jan. 15, 1919, have declared that the tank was safe although subjected to stresses of about 30,000 lb. per sq.in., that numerous other molasses tanks of the company along the Atlantic seaboard are more highly stressed and yet stand, and that all appearances suggest that a dynamite explosion wrecked the tank. F. E. Sherry, Prof. George F. Swain, Prof. George E. Russell, and L. E. Moore have testified for the defense. The defendant corporation has pleaded in defense of the damage suits that the failure was due to conditions beyond its power to control.

According to the testimony of the expert witnesses the tank, although stressed higher than might be desirable, was not stressed to the elastic limit of the material. From tests made on specimens of the material, the ultimate strength was found to be 55,000 lb. per sq.in., the elastic limit 33,000. The stress near a manhole where a vertical crack was found after the accident, was set by Prof. Swain and Prof. Russell at between 28,000 and 29,000 lb. per sq.in. Slightly higher stress existed at some other points. Metallurgists for the defense testify that the steel shows signs of severe shock and has no indications of a gradual drawing out and thinning down of the material. One witness living near the site of the tank testified to seeing a light smoke rise from the ventilator at the top of the tank a few minutes previous to the accident. Experts testify that this smoke can be explained only by the presence of a burning fuse.

Price Trend and Supply in the Cement Industry

By Owen M. Fox
Construction News Department, *Engineering News-Record*

IN VIEW of the severe shrinkage in the construction industry many cement users have been led to the belief that shortage conditions prevailing for some months past should soon be overcome and that possibly lower prices might be imminent. Inquiry in the industry does not bear out this idea.

While it is true that construction is at a low mark all over the country it should be remembered that a larger proportion of cement is used in construction now than formerly. The greatest pressure of demand, however, comes from dealers supplying rural communities and agricultural districts. In such territory, small jobs which have been deferred for long periods on account of either war restrictions or prior to the war, lack of money, are now going ahead, and these consuming units, small individually, form an enormous total.

These reasons explain why although production is now on a greater scale than it was last year, it is below demand. Car supply, so great a factor in the past, is gradually improving, but other obstructions, such as uncertain coal supply or inefficiency or scarcity of labor, still tend to keep most plants running with an output below capacity.

Prices have recently advanced to cover the increase in freight rates, effective Aug. 26, and the outlook gives no indication of an early reduction. Certain producers are paying from 75 to 85 per cent more for their coal, on contract, than formerly. In many instances only a fraction, approximately one-third, of their need is being supplied under contract and to supply this deficiency coal has been purchased in

the open market in competition with speculators and manufacturers of long-profit lines, at an advance over contract price of from $3 to $8 per ton. Furthermore, such open market purchases are frequently below the requirements in B.t.u. for the proper burning of cement to such an extent that 25 or even 50 per cent more coal is required to produce a specific quantity of finished product. Taking into consideration the fact that every dollar added to the cost per ton of coal increases the actual cost of cement five cents per barrel, it is seen that this cost has been materially raised by the prevailing coal hysteria.

Other features also add to the production cost. Car shortage means that producers must ship in whatever rolling stock is available. Much cement has been shipped in privately owned refrigerator cars, at a minimum freight charge of 50,000 lb., when the actual capacity of the car is much less, and at the expense of a stiff rental paid to the car owners. Cases are frequent of such excess charges amounting to $25 or more on a car load, all of which must be added to the cost of cement to the ultimate consumer. Large sums have been spent in labor cost for cleaning stock cars which have been assigned by the railroads to cement hauling and the already troublesome labor factor has been intensified by men being required to perform this unpleasant and unexpected work.

We entered the building season this spring with reserve stocks far below normal and a multiplicity of factors have kept plants running at less than capacity so that there has been no chance to overcome the deficiency. Effort is being centered now on securing quantity production rather than economy. Should we pass the next six months with fuel, transportation and labor suffering no serious derangement, the 1921 season will open with ample stocks and some possibility of lowered prices. For the present each week sees some improvement in the car situation. Each week brings us nearer to a return to reason as regards coal prices. Each week therefore sees a slight improvement in the cement situation. But we have a long road to travel before supply equals demand and before any downward revision of price scales can be expected.

$60,000,000 Needed for River and Harbor Work

Government river and harbor improvement work now is being carried forward at the rate of $3,500,000 monthly. This is exclusive of the Wilson Dam, where the monthly expenditure is $700,000.

Since the next appropriation bill will have to cover a period of 16 months, and as it is considered desirable to continue work at the present rate, it is expected that the next Congress will be asked to appropriate some $60,000,000 for river and harbor improvements.

Edwin Thacher

Edwin Thacher, one of the pioneers of reinforced concrete in America, former chief engineer of the Keystone Bridge Co., known the world over as originator of the Thacher slide rule, inventor of the Thacher truss, the first deformed reinforcing bar, and an improved system of arch reinforcement, died Sept. 21, 1920, in New York City, at the age of eighty-one. He passed practically his entire professional life in bridge engineering, the earlier half in iron bridge construction and the later half in reinforced concrete. The

EDWIN THACHER

most notable achievements of his career were those of the former period; they include, besides the development of the slide rule and the truss which bear his name, a study of column strength leading to the deduction of the straight-line formula (in the publication of which, however, he was anticipated by Thomas H. Johnson, who had been independently studying the subject at the same time as Mr. Thacher), the preparation of extensive tables of stress coefficients for bridges, and the aggressive advocacy of rational detailing in iron bridge construction. Particularly through his stand for good details of design, including opposition to the then prevailing practice of using eccentric connections such as top-chord pin connections at mid-depth of a U-chord, Mr. Thacher exerted a strong and lasting influence on the development of bridge designing practice in the United States. His influence upon the younger men who worked with him was heightened by a generous and human disposition and a high character.

Born Oct. 12, 1839, at DeKalb, N. Y., Edwin Thacher obtained the degree of civil engineer at Rensselaer Polytechnic Institute in 1863. During the next five years he was successively assistant engineer of the Cedar Rapids & Missouri River R.R., of the U. S. Military Railroads, and of the Louisville, Cincinnati & Lexington R.R. Entering then the field of iron bridge building, he remained in this activity for more than thirty years. After two years as resident engineer of the Ohio River bridge at Louisville (recently rebuilt), he went to the Louisville Bridge & Iron Co (1870-80), became calculating engineer for the Keystone Bridge Co., of Pittsburgh (1880-83), and rose to the position of chief engineer of the company. In 1887 he left Keystone to become chief engineer of a bridge works at Decatur, Ala., which however failed soon afterward. Returning to Louisville he practised for five years as consulting engineer and bridge contractor (1889-94) and then formed a partnership with Henry Keepers, of Detroit, for similar practice (1894-99). During this period Mr. Thacher became interested in the Melan system of reinforced concrete arch construction, then being introduced by F. von Emperger, and took up the representation of this system in the West. In this capacity he developed the final designs for the Topeka reinforced-concrete arch bridge, a structure which as the largest of its type in the world marked an epoch in the development of concrete construction; and the firm of Keepers & Thacher secured the contract for the bridge and built it. From 1899 to 1901 as member of the firm Thacher & Conners he built two important bridges for the United Sates military government in Porto Rico. In 1901 he combined with William Mueser as the Concrete Steel Engineering Co., to develop jointly the Melan, von Emperger and Thacher methods of reinforced-concrete arch construction. He continued actively in this firm until 1912, since which time he has done only occasional professional work, finally retiring two years ago.

The widely used multiscale cylindrical slide-rule, with double 30-ft. scale, was devised by Mr. Thacher during or shortly after his Louisville period (being patented Nov. 1, 1881). Some years later he developed a 24-in. flat rule with improved scale arrangement to keep the result on the rule at all times. While still at Louisville prior to 1880 he had begun the calculation of tables of stress coefficients for all current forms and proportions of bridge truss, including both iron, wood and combination. As finally used these tables included the Thacher truss, developed while he was at Keystone; it was a highly efficient form of partly single and partly double intersection truss, adapted particularly to combination wood and iron construction and considered to be largely free from thermal stresses. The Thacher reinforcing system for concrete arches was devised about the time he first interested himself in the Melan system, and his deformed bar was invented some years later. Mr. Thacher's work for the improvement of bridge design methods was at its height during his service at Keystone. His influence on this establishment contributed in a significant way to bringing the art to its present-day excellence of practice.

ENGINEERING SOCIETIES

Calendar

Annual Meetings

AMERICAN SOCIETY FOR MUNICIPAL IMPROVEMENTS, Valparaiso, Ind.; St. Louis, Oct. 12-15.
AMERICAN RAILWAY BRIDGE & BUILDING ASSOCIATION, Chicago; Atlanta, Ga., Oct. 26-28.
NATIONAL DRAINAGE CONGRESS, Chicago; Atlanta, Ga., Nov. 10-12.

The Brooklyn Engineers' Club held a a special meeting Sept. 23 for the purpose of discussing the invitation extended the club by the Joint Conference Committee to become a charter member of the Federated American Engineering Societies. Richard L. Humphrey, chairman of the committee, was present and addressed the members.

The Rochester (N. Y.) Engineering Society, at its first fall meeting, Sept. 10, was addressed by Col. Samuel P. Moulthrop, whose subject was "Indian Life In and Around Rochester." The talk was illustrated with colored slides and dwelt particularly on the various trails passing through or near the city, as the Indian was the forerunner of the engineer in establishing lines of communication. The following officers have been elected for the coming season: President, Henry L. Howe, engineer in charge of city testing laboratory; first vice-president, Harold O. Stewart, production engineer for Max Lowenthal & Son; second vice-president, Charles C. Evans, architect; secretary, Joseph W. Ward, works engineer, Taylor Instrument Co.; treasurer, Louis J. Summerhays, of William Summerhays & Son.

The Engineering Society of Buffalo, including its affiliated bodies, on Sept. 21 unanimously voted to become a charter member of the Federated American Engineering Societies. Members of the Automotive Engineers Society, the American Steel Treaters Society, American Institute of Architects, American Chemical Society, American Society of Mechanical Engineers and the Electrical Engineers Society were present at the meeting. A project was discussed to incorporate the Engineering Society of Buffalo for the purpose of organizing the members of the various affiliated bodies on a united front for the discussion and solution of important engineering problems confronting the city and the western district of New York. Dexter S. Kimball, dean of the engineering school of Cornell University, and Richard L. Humphrey, consulting engineer, Philadelphia, and chairman of the Joint Conference Committee, addressed the meeting on the subject of the federation. Among the topics to be discussed during the fall and winter are the Niagara Falls hydro-electric development.

PERSONAL NOTES

PROF. FREDERICK H. NEWELL, head of the department of civil engineering, University of Illinois, and formerly director of the U. S. Reclamation Service, has resigned his position. Professor Newell was president of the American Association of Engineers last year and this summer acted as director of the field forces.

GEORGE C. BUNKER, in charge of water supplies on the Canal Zone, is now in the United States. He has recently visited Bogota, Colombia, where typhoid is endemic, to investigate the water supply for the board of sanitation, and in November will return to that city to install ten water sterilizing outfits.

WILLIAM EARL RUSS & CO. is the name of a new firm of architects and engineers organized, at Indianapolis, by William Earl Russ, architect, of that city, formerly with Frank M. Andrews of New York; Walter H. Scales, formerly chief designing engineer with the Lackawanna Bridge Co., at Buffalo, for the past year associated with Mr. Russ as chief of engineering and construction; and Ralph J. Batcheldor, formerly with Allen & Collens, Boston, and Holabird & Roach, Chicago.

A. R. JONES, supervisor of tracks, New York Central Lines East of Buffalo, with headquarters at Clearfield, Pa., has been made division engineer of the St. Lawrence Division, at Watertown, N. Y., succeeding P. H. Winchester, transferred.

MERVIN E. LYLE, formerly assistant to the president of the Columbia Graphaphone Co. and for a number of years manager of the company's Bridgeport, Conn., factories, is now associated with Willard C. Brinton, consulting engineer, of New York, and has been appointed assistant to the president, Terminal Engineering Co., Inc., of which Mr. Brinton is president.

OSCAR ZOLOAGA, hydraulic engineer, of Venezuela, who is at present in the United States, recently inspected the Miami Conservancy District, at Dayton, Ohio. Mr. Zoloaga is constructing several dams in his country.

E. J. STEPHENSON, of Minneapolis, has been appointed civil engineer for the United Light & Railways Co., Davenport, Iowa.

WOUTER COOL, civil engineer, of Rotterdam, Holland, and Batavia, Dutch East Indies, is visiting the United States, on his way from Batavia to Holland, with a view to studying the dock and wharf facilities of this country. He has examined the waterfronts of San Francisco, Seattle, Vancouver and Portland and expects to make similar studies at New York and Boston. For the past five years Mr. Cool has been general harbor advisor to the Netherlands East India Government. Upon his return to Java, it is announced, he will be also director general of the East Indies railroads. Mr. Cool is accompanied by U. S. RONDE BRESSER, dredging expert and general contractor, who constructed the harbor works at Soerabaja, and has handled several other large government contracts.

LLOYD ALDRICH, recently tendered his resignation as engineer of Sonoma County, Cal., effective Oct. 31.

F. C. FINKLE, consulting engineer, Los Angeles, has been appointed consulting engineer for the Victor Valley Irrigation District, comprising more than 70,000 acres in San Bernardino County, Cal., to supervise the design of an irrigation system for the district.

R. E. SPEAR, formerly engineer accountant for the Illinois Central R.R. Co. in the bridge, building and waterworks department, has accepted the position of borough engineer of Ambridge, Pa., which is operating under the city manager form of government with W. M. Cotton as borough manager.

ARTHUR M. SHAW, consulting engineer, New Orleans, will have general direction of the newly established civil engineering department of Loyola University in that city. He will also continue his office for private practice.

OBITUARY

JULIUS REICHEL, formerly civil engineer in the Carbondale, Ill., office of the Illinois State Highway Department, died recently at Havana, Ill.

CAPTAIN WALTER C. PATON, Corps of Engineers, U. S. A., formerly city engineer of Excelsior Springs, Mo., died in that city Sept. 11, at 38 years of age. He formerly lived in Kansas City and was chief inspector in the construction of the inter-city viaduct.

WILLIAM G. BUSSLER, chief engineer of the Waterloo (Iowa) waterworks, died Sept. 12, in that city. He was born near Waterloo, Sept. 30, 1864, and had been employed at the waterworks for 32 years.

HUGH B. RAINES, who was city engineer of Dallas, Tex., for twenty years, retiring several years ago when the commission form of government was adopted, died at Louisville, Ky., Sept. 17. Mr. Raines had lived for 45 years in Dallas and was 70 years old. During his service as city engineer he laid out and superintended the construction of some of the most important engineering works in Dallas.

S. D. PORTER, for the past twenty years city engineer of Peru, Ill., died Sept. 12, at LaSalle, Ill. He was born in Hatfield, Mass., Jan. 1, 1850, and studied engineering at East Hampton, Mass. Previous to his service with the City of Peru, he was for ten years connected with the Illinois Central R.R.